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NON-BITUMINOUS ROAD MATERIALS.[1]

INTRODUCTION.

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CONTENTS.

	PAGE
Introduction	557
Methods of Investigation	559
Bedrock Deposits	560
Field Methods	560
Service Tests	561
Laboratory Tests	565
Sampling	566
Value of the Tests	567
Petrographic Work	575
Comparative Value of the Different Classes of Rock	575
Glacial Boulder Deposits	580
Gravel Deposits	583
Field Methods	584
Laboratory Tests	585
Value of the Tests	590
Summary	594

With the increase of the mileage of improved country highways in the United States and Canada, there has come into being a new profession, that of highway engineering. The highway engineer has been called upon to solve the problem of constructing and maintaining road surfaces to accommodate a traffic, that has

[1] Published by permission of the Directing Geologist of the Geological Survey of Canada.

not only grown enormously within the last decade, but has rapidly changed in character. The destruction of road surfaces by heavy swiftly moving motor vehicles has forced the adoption of new methods of construction and new materials, involving the profession in researches along chemical and physical lines. Within the last few years the need for geological work along these lines is also becoming apparent. The demand just now far exceeds the effective supply.

The mass of practically all country highways is made up of natural rock or gravel aggregates. In the waterbound macadam road surface, fragments of broken stone or of gravel are bound together by the products of the reaction of rock dust and water under pressure. Under modern heavy traffic conditions this surface fails rapidly, and engineers have been compelled to use artificial binders, such as bitumen and Portland cement to reinforce the surface. The large sums of money involved in the construction of roads containing these products, together with keen commercial rivalry among the firms engaged in their manufacture, has given rise to chemical and physical investigations which have brought about great improvements in the quality of the manufactured binders. Accompanying this there has been an improvement in the methods of road construction, until today both engineers and manufacturers realize that future progress must come from a more careful selection of the rock or gravel aggregate, that forms the bulk of the road, and whose quality may make or mar a road in spite of highly specialized binder products and careful construction. By far the larger proportion of country roads, too, are of waterbound macadam or sand clay, in which the so-called "mineral aggregate" is of even greater importance. The demand for the investigation of deposits of road stone and gravel as a preliminary to road construction is, therefore, becoming more frequent and wherever possible, the engineer is leaving this work to the geologist.

The business of building roads is moving apace and many geologists may find themselves involved in work that demands a knowledge of the road making values of stone or gravel with all

too brief a time for preparation. From personal experience of that kind the writer is led to believe that a review of the methods of field and laboratory investigation together with a summary of what is known of the criteria for determining the better classes of road stone and gravel will be of interest.

This paper is written after four years of field and some laboratory experience. It is intended to serve as an introduction to the study of the so-called non-bituminous road materials,[2] namely, stone and gravel.[3]

The writer is indebted to his colleague, Dr. K. A. Clark, of the Department of Mines of Canada, who planned and carried out most of the experimental work undertaken by the road materials division, and has rendered valuable aid in the preparation of this paper.

METHODS OF INVESTIGATION.

Road material surveys may be made to discover in a general way the principal sources of road material within a certain district and their relative values, or they may be for the purpose of obtaining specific information regarding the local deposits available for a certain proposed highway or system of highways. The latter, is, to the writer's mind, the more important of the two. For, while a general review of the available road materials of a district is instructive, the variations that may occur in the same class of materials in a district are too great to make such surveys of direct value. It should be the object of detailed surveys on the other hand to furnish road builders with information which will enable them to obtain efficient materials at the lowest possible cost. The construction of a highway is a costly undertaking and

[2] Stone and gravel are the principal non-bituminous road materials that occur as natural deposits. Artificial non-bituminous road materials such as slags are not discussed in this paper.

[3] For further information the reader is referred to the many excellent publications of the Office of Public Roads at Washington, which are freely quoted here, to the *Proceedings* of the American Society for Testing Materials and other engineering bodies, to the numerous treatises on highway engineering and the periodicals devoted to highway or general engineering.

a large proportion of that cost[4] is spent in hauling the required crushed stone or gravel. The exact location of all deposits of available material near a proposed road is, therefore, of great importance to the road builder. The gathering of data regarding the quantity of stone or gravel that is available, and the comparative cost of quarrying, crushing, and transportation, should be collected in the field. The quality of the material is, however, as important from an economic standpoint, since the use of stone or gravel that is not of sufficiently durable quality for the road in question, may increase the cost of maintenance by several hundred dollars per mile per year. The special duty of the geologist will always be to determine the value of the stone or gravel itself, and to do so effectively it is necessary to arrive at field criteria for determining a good road-making stone or gravel. These criteria are discovered from field studies both of the rock or gravel in place and of its action in such roads as have been made from it, and by laboratory tests of the physical properties of samples of the deposits, combined with petrographic studies of the stone tested.

The natural non-bituminous highway materials may be conveniently classified into bedrock, boulder and gravel deposits. In the following pages the methods of investigating each of these is discussed separately and in more detail.

BEDROCK DEPOSITS.

Field Methods.

The general field methods discussed above are applicable to bedrock deposits.[5] The writer would, however, emphasize the

[4] The average cost per mile of a waterbound macadam road 16 feet wide and 6 inches thick, for instance, ranged from about $7,000 to $10,000 under pre-war conditions; to-day, it is perhaps, 30 per cent. higher. It takes a little more than 2,000 cubic yards of crushed stone or gravel to build a mile of such road, and if the average wagon haul of this material from pit or crusher to construction point be 2 miles, the cost of haul averages about 16 per cent. of the total, with an increase of roughly 8 per cent. for every increased mile of haul.

[5] A detailed schedule for use in the field examination of bedrock deposits of road material is to be found on pages 136 to 137 of the "Handbook for Field Geologists," by C. W. Hayes.

importance of obtaining details regarding the mineral composition and texture of the stone examined, the type and extent of alteration to which it has been subjected, and all the information possible regarding the fracture and cleavage planes present. The need for such observations is brought out in discussions that follow upon the values of the toughness test, and upon the factors that govern the road-making qualities of the various classes of stone.

If no geological map of the area surveyed is available the various formations should be separated on the map wherever that is possible. The stone within a formation can then, if necessary be further divided into lithological units.

It has been proved by numerous laboratory tests upon a series of limestone formations that, in certain cases at least, there is a marked similarity in the quality of stone from the same formation and that the average of one formation differs from that of another occurring close by and made up of the same class of rock.[6] The average quality differs even more when the lithology of the formations vary. One of the most useful results of the mapping of formations is that outcrops lying within whole areas can in many cases be definitely rejected as unfit for roadwork, and the time so saved spent in detailed prospecting of areas underlain by the more promising formations.

Service Tests.

The efficiency of a road surface, its comparative smoothness and the length of time that it will remain in good condition, depends on several factors, of which the quality of stone comprising its surface is only one, and not in every case the most important. By surface is meant here, the layer, usually 4 to 8 inches thick that takes the wear of the traffic. The chief of these factors are the volume and intensity of the traffic, the strength of the foundation upon which the surface rests, the manner in which the surface itself has been constructed, and the effectiveness with

[6] Reinecke, L., and Clark, K. A., "The Sampling of Deposits of Roadstone and Gravel in the Field," *Proc. Am. Soc. Test. Mat.*, Vol. XVIII., 1918 (in press).

which water is drained away both from the foundation and the top of the road. Climate and other minor influences also play their part. Before a geologist can attempt to use the condition of a road as a criterion for determining the value of the stone of which it is made, he must be able to determine in how far these factors have affected the road he is examining. To understand their influence he should not only read some of the many treatises on highway engineering, in all of which attention is drawn to them, but try to learn as much as possible regarding the details of road construction from actual observation.

Wherever careful observations upon the wear of stone in a roadbed are to be made a census of the traffic passing over it is an essential part of the work. The effect of traffic depends on its character; the steel tires of wagons have an abrasive, while the rubber tires of automobiles have a shearing action upon the surface; both subject the inside of the road to abrasion and impact. The actual weight of the loads per unit width of tire, and the number and weight of vehicles passing within a certain period, that is, the intensity of traffic, are as important as the total weight passing in a longer period of time.

Highway engineers recognize that it is not the layer of surfacing material on a road but the earth base below it that carries the loads of traffic. The surfacing layer is generally in the form of an arch, but even when fairly rigid the arch is too flat to support the loads imposed on it. It serves to distribute the load over a larger area of base, to provide a smooth surface for traction and to prevent water falling on the surface from penetrating to the subbase. If the subbase cannot support the traffic and becomes permanently deformed the surfacing layers eventually yield also, and the road surface is destroyed irrespective of the quality of the stone comprising it. To prevent this an artificial foundation of greater or less thickness and rigidity is placed between the earth base and the surfacing layers where the traffic and condition of the earth base require it. Such a foundation serves to arch over small weak areas, and to distribute the load over a much larger surface of the base. The first point to be observed therefore is whether that portion of the road lying under

the wearing course or courses is rigid enough to bear traffic without permanent deformation.

The method of constructing the surface courses of a water-bound macadam road should be such as to ensure a mass in which the voids in the upper layer are at a minimum and the top surface is as smooth and resilient as possible. Proper sizing of the stone and adequate rolling and sprinkling during construction tend toward this condition. Carelessness in the application of any of the details of construction may cause a failure of the surface as rapidly as the quality of the stone itself. Longitudinal ruts in a macadam road may for instance be caused either by inadequate rolling or the softness of the stone in the surface.

Effective under drainage is necessary because a wet subbase is not as strong as a dry one and the freezing and consequent heaving of a wet base in cold weather may destroy the surface above it. It is necessary to prevent water from standing on a road because the surface is softened and there is more shearing and abrasion of the surface of a wet, than a dry road. It follows also that water standing in holes in a road tends to the local enlargement of the holes and the soaking of the water through to the subbase. If a stretch of road surfaced with a certain type of stone have a sufficiently resistant foundation, adequate drainage and is properly constructed, it should form the best possible means for arriving at the road making value of the stone. By observations covering a period of years it is possible to discover the length of time under which the road surface will remain "in good condition" under given traffic conditions.

Unfortunately there are few quantitative means of determining the degree of deterioration of the surface from year to year. The amount of tractive power required to draw a certain load over different types of road surfaces at various speeds has been determined by specially constructed trucks[7] and by the same means the increase of power required from year to year could probably be arrived at and used as a measure of its deterioration.

[7] Kennedy, A. E., and Schurig, O., "Tractive Resistances to a Motor Delivery Wagon on Different Roads at Different Speeds."

In France[8] a number of service tests were made in the following manner. All the materials used in the construction and repair of some 22,000 miles of national road were recorded, a comprehensive traffic census was kept and each divisional engineer was required to build short sections of road of each available rock in his district along the same lines of travel. In 1874, 1886 and 1891 the depths of these roads were measured by digging trenches at intervals of 200 meters on alternate sides of the roads from the side to the center. From these measurements the amount of material worn off per mile per year for a certain number of units of travel was calculated. These tests probably formed a very efficient means of determining the relative values of the various deposits of stone used on the national roads of France but there are numerous difficulties in the way of this being undertaken in Canada or the United States. Model roads have been built in short sections along the same lines of travel in Great Britain and the United States to test various kinds of road materials and different methods of construction. Several forms of machines for testing road surfaces have also been tried. In the laboratory of the Road Board[9] of England a road testing machine has been in use since 1913. In this test the road material is laid on a concrete foundation on a circular path 34 feet in diameter and 2 feet in width. Attached to the frame work of the testing machine are eight wheels 3 feet 3 inches in diameter with 3-inch tires. These travel around on the circular track each wheel driven by its own motor and loaded with weights of from 470 to 750 lbs. a piece. The track surface is usually kept wet by spraying and the material worn off collected and weighed. The number of tons of "laboratory traffic" required to abrade a certain amount of material can be computed from the results.

Machines on the plan of the brick rattler are used in the United States for testing the resistance to abrasion of sections of road concrete.[10]

[8] Page, L. W., "Annual Report of the Geologist—Road Materials and Their Physical Properties." Seventh Annual Report Massachusetts Highway Commission," Boston, January, 1900, p. 86.

[9] *The Quarry*, London, England, September, 1916, p. 200.

[10] Abrams, D. A., "A Method of Making Wear Tests on Concrete," *Proc.*

The discovery of effective methods for testing road surfaces rather than the aggregates composing them would prove of immense value to road engineers.

Laboratory Tests.

The principal tests[11] used in the laboratory to-day are those for abrasion, toughness, hardness, cementing value, specific gravity and absorption (Plate XL.).

In the abrasion test about 50 pieces of the stone weighing within 10 grams of 5 kilograms are shaken back and forth by the revolution of a cylinder mounted at an angle of 30° with the axis of revolution (Plate XL., *A, a*). The weight of material passing $\frac{1}{16}$-inch mesh that is worn off in 10,000 revolutions, expressed as the percentage of the weight of the charge, constitutes the measure of the abrasion of the stone. It is known as the per cent. of wear. Wear is also expressed by the so-called "French coefficient of wear," an arbitrary factor derived by dividing 40 by the per cent. of wear.

In the toughness test the blow due to the fall of a 2 kilogram hammer is transmitted to a cylinder of the stone to be tested through a plunger with a spherical end of a radius of 1 centimeter (Plate XL., *B, f*). The cylinder, 25 millimeters long and of the same diameter, is bored with a diamond core drill (Plate XL., *A, f*) out of a block of the stone to be tested and the ends are sawn and ground to the required length. It is important that the ends of the cylinder be plane and at right angles to its length. Successive drops of the hammer are registered beginning with a

Amer. Soc. Test. Mater., Vol. XVI., 1916, Part II., pp. 194–208. Nash, "Tests of Concrete Road Aggregates," *Proc. Amer. Soc. Test. Mater.*, Vol. XVII., 1917, Part II., pp. 394–421.

[11] For detailed description of apparatus and methods used in these tests the reader is referred to Bulletin 347 U. S. Dept. Agriculture, "Methods for the Determination of the Physical Properties of Road Building Rock," Frank H. Jackson, Jr., Washington, D. C., 1916. The abrasion and hardness tests were first used in the French School of Bridges and Roads, the toughness and cementing value tests were designed by Mr. Logan W. Page, director of the Office of Public Roads, Washington, D. C., who initiated the testing of road materials in the United States, and has been chiefly responsible for their subsequent improvement.

height of one centimeter and increasing in height by one centimeter per blow until the cylinder fails. The height of fall of the hammer as which the cylinder fails, in centimeters, is taken as a measure of the toughness of the stone.

In the hardness test (Plate XL., *A, e*) cores of the same diameter as for the toughness test are held vertically under a constant load against a revolving disc charged with a special quartz sand. The loss in grams after 1,000 revolutions is divided by 3, and the result subtracted from 20 is used as a measure of the hardness. Cores are drilled from the same block as those to be used in the toughness test.

In the test for cementing value 500 grams of the stone broken to pass a ½-inch screen is ground with 90 cubic centimeters of water for 5,000 revolutions in a special ball mill (Plate XLI., *A, d*). The dough is removed and moulded into briquettes 25 millimeters in diameter and 25 millimeters in height. The hydraulic briquetting machine is of special design (Plate XLI., *A*), and imposes a maximum pressure of 132 kilograms per square centimeter on the briquettes during the process. After drying for 20 hours in air at room temperature and four in an air bath at a temperature of about 100° C. (Plate XL., *B, a*), the briquettes are cooled in a dessicator and broken in an apparatus where it is subjected to the blows of a one kilogram hammer falling through a vertical distance of one centimeter (Plate XL., *B, e*). The number of blows required to destroy the resilience of the briquette is taken as a masure of the cementing value of the stone.

The absorption is expressed in the pounds of water per cubic foot of stone absorbed by a fragment of stone during a total immersion of four days.

Sampling.

In sampling, pieces of stone, as nearly cubical and close to 2 inches in width as possible, should be broken at equidistant points across the strike of the beds, if sedimentary, or across zones of variation, such as those of texture, if igneous, and only fresh stone should be taken. Since at least 50 pieces are required for the abrasion test and more in certain cases, the number should

A.

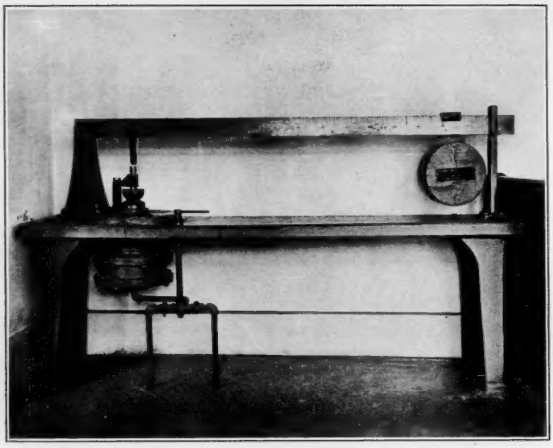
Portion of the road materials laboratory, Mines Branch, Department of Mines, Canada.

Explanation: *a*, Deval abrasion machine with *b* charge for abrasion before and *c* after the test; *d*, Ball mill; *e*, Dorry hardness machine; *f*, core drill.

B.

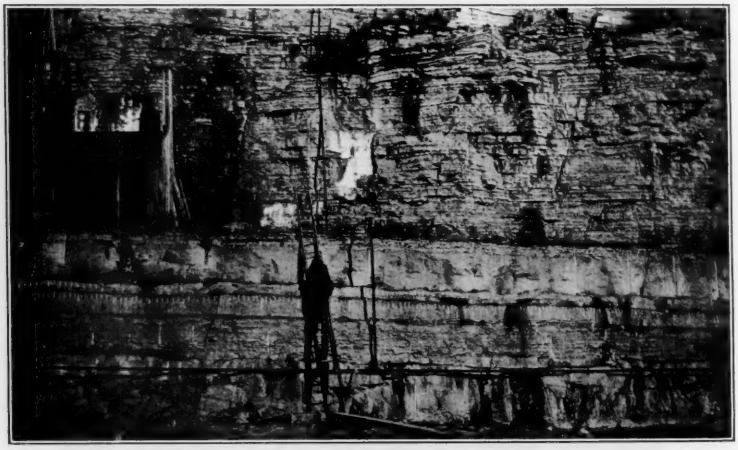
Another portion of the laboratory.

Explanation: *a*, electric drying ovens; *b*, set of Tyler sieves; *c*, cone for testing voids; *d*, diamond saw and grinding lap; *e*, impact machine for testing cementation; *f*, Page impact machine for testing toughness.



A.

Hydraulic press for moulding briquettes, cementation test.



B.

Wall of Trenton limestone, Martineau's quarry, Montreal, Que. The two beds with many short drill holes at man's waist have a toughness of 5, the massive beds above and below this of 6, and the upper thin beds of 7.

not be less than 55 and the fragments not less than 2 inches in diameter. To this sample should be added a block or number of blocks, about $6 \times 4 \times 3$ inches in size, broken wherever possible by the plug and feather method. From these the cores used in the toughness and hardness tests are to be drilled. To facilitate the drilling of cores the blocks should be as nearly rectangular as possible. One block is unsatisfactory, as an average of the toughness value of the whole deposit is not usually obtained from it; three or more blocks are better, to represent variations not only in the lithological character of the deposit but in the structural peculiarities of its different parts.

In sampling quarry faces care should be taken not to take stone from that portion lying immediately adjacent to the location of holes used in blasting, as the rock fragments so taken may have been weakened by the blast.

Value of the Tests.

The value of these tests depends on many factors: the degree of similarity between the tests imposed upon fragments of stone in the laboratory and those they undergo under the action of traffic in the roadbed; the amount of variation in the results arising through the operations of sampling and laboratory manipulation; and the closeness with which the samples taken correspond in their roadmaking value to the deposits they are supposed to represent.

Abrasion Test.—The abrasion test is for many reasons probably the most satisfactory of the laboratory tests. It resembles the action that takes place in a macadam road surface, and the method of sampling and testing is such that the result obtained in the sample approaches the average for the whole deposit. Much abrasion takes place in the surface of a macadam road, for the interstices between the larger stone fragments are found filled with small stone chips and fine mud, and the stone fragments themselves have rounded edges; and the more worn the road, the larger is the proportion of mud and small stone. When first placed in the roadbed the stone has sharp edges. Those pro-

jecting at the surface are soon smoothed down by the direct abrasion of the wheels passing over them, while the stones under the surface abrade each other during the slight movement that takes place in the mass of the road when loads pass over it. In the abrasion machine the edges of the stone fragments are worn off as the pieces are thrown from one end of the cylinder to the other, and the result is a number of rounded stones and a mass of fine dust (Plate XL., *A, c*).

The resistance to abrasion of a stone appears to depend upon the resistance of the individual grains and their cohesion rather than upon the cohesion along planes of weakness between one part of the stone and another. In other words resistance to abrasion is an "inherent" character of the stone, that is apt to be similar throughout unweathered rock of the same lithological character, and is largely independent of what might be termed the "accidental" variations in strength, due to incipient cleavage planes in the rock. The result of the abrasion test is an average of the resistance to abrasion of 50 possible variations in the lithological make-up of the deposit. It is not surprising, therefore, that the relative results of the tests should be closely parallel to the relative resistance to abrasion of the deposits themselves under the action of traffic.

The significance of the results of the abrasion test depends to a great degree also on the possible variations in results of the tests that may arise from causes other than the quality of the stone itself. The following results of experiments by Clark[12] upon the possible variations due to laboratory manipulation and sampling and the variations between the rock in place and the crushed product are therefore of interest.

1. The results of an abrasion test expressed in per cent. of wear is liable to an error of 0.2 due to variations caused by laboratory proceedure.

2. No additional error is introduced into the results by the process of sampling when samples are collected as outlined above.

[12] Reinecke, L., and Clark, K. A., "The Sampling of Deposits of Roadstone and Gravel in the Field," *Proceedings of the Amer. Soc. Test. Mater.*, Vol. XVIII., 1918, Part II. (in press).

3. As affecting the number of samples that are required to arrive at the average per cent. of wear of a rock deposit, it was shown that in stone from a section of sedimentary beds of uniform lithological character along the strike though varying from bed to bed across the strike, and in an igneous body of uniform lithological character, the per cent. of wear of samples taken in one part of the deposit can be regarded as representing within the error due to laboratory manipulation, the per cent. of wear throughout a fairly large mass of the deposit. This was proven for a section of dolomitic beds 50 feet thick over a bedding plane area of 160 acres (Plate XLII.) and for a sheet of tinguaite 25 to 40 feet in thickness and of the same areal extent.

4. Results of abrasion tests on rock in place in a deposit represent within a probable difference of 0.4 the per cent. of wear that will be shown by a crushed product from the same deposit.

These results hold true when the sampling and testing are done carefully by the same operator in the same laboratory. The results from two different laboratories may vary considerably.

The range in percentages of wear as determined by this test upon a great many samples of stone suited for road building runs from about 1.5 to 8. Just what the equivalent of per cent. of wear is translated into the volume or mass of traffic that a roadbed made of a certain stone can carry is hard to state within narrow limits. The following are amongst the latest recommendations regarding the suitability of stone of a certain per cent. of wear for varying traffic conditions. The recommendations in regard to toughness are for convenience included with it.

Toughness Test.—Toughness is defined by Page[13] as the power

[13] Page, L. W., "Road Materials and Their Physical Properties," Seventh Annual Report of the Massachusetts Highway Commission, pp. 69, 70, January, 1900. It is of interest to point out, as Page has done, that in substances that are perfectly homogeneous, structureless and incapable of permanent deformation without fracture, the energy of the blow causing fracture is proportional to the maximum compressive stress at the point of fracture and also to the accompanying deformation or strain. In other words the toughness test can be used as a measure of a definite property of such a substance. This property, depending on the strength of the individual particles composing the substance and the cohesion between the particles, may be referred to as the elasticity or "inherent toughness" of the substance. Practically all road-

LIMITING VALUES OF PER CENT. OF WEAR AND TOUGHNESS UNDER VARYING TRAFFIC CONDITIONS.

Type of Road-surface.	Light Traffic up to about 100 Vehicles per Day.		Moderate Traffic 100 to 250 Vehicles per Day.		Heavy Traffic 250 Vehicles per Day and Over.	
	Per Cent. of Wear.	Toughness.	Per Cent. of Wear.	Toughness.	Per Cent. of Wear.	Toughness.
Water bound macadam: *a*....	5 to 8	5 to 9	2.7 to 5	10 to 18	0 to 2.7	over 18
b....	5.7−	6+				
c....	5−	6+				
Bituminous broken stone with seal coat and broken stone with bituminous carpet: *a*..	8−	5+	5.7−	10+	5.7−	10+
Bituminous concrete with or without seal coat: *a*.....	5.7−	7+	4−	13+	4−	13+
b.....	3.5	13+	3.5−	13+	3.5−	13+
c.....	3.5−	13+	3.5−	13+	3.5−	13+

[a] Recommended by the Office of Public Roads, Washington, 1916.
[b] Recommended by the American Society of Municipal Improvements, 1914.
[c] Recommended by the American Society of Civil Engineers, 1917.
+ = "and over," that is, the figure is a minimum value.
− = "and under," that is, the figure is a maximum value.
The recommendations *b* and *c* simply give minimum limits for toughness and maximum for per cent. of wear for the type of road in question.

possessed by a material to resist fracture when subjected to impact. The stone in the wearing course of a roadbed is continually subjected to the impact of horses' hoofs and more especially to that caused by the bumping of the wheels of swiftly moving motor vehicles passing over small obstructions. The relative resistances to impact of one inch cylinders of stone are evidently measured with efficiency by the laboratory impact machine, and the value of the test will therefore depend upon how closely the toughness values of the cylinders, which under present practice are derived from a single small block of stone, approach the average toughness values of the thousands of fragments of stone in a roadbed, representing as they do all parts of the deposit from which they are derived. The toughness value of one $6 \times 4 \times 3$ block cannot, however, be expected to represent the

stones under the conditions of the test are incapable of deformation without fracture. If deposits of such stone were amorphous, or made up of sufficiently fine grains of the same size, without lines of weakness, the toughness of one part of such a formation would be the same as another irrespective of the direction from which the force causing fracture is applied.

average value of a stone deposit, because the toughness values vary with variations of mineral composition and texture within the deposit, and they also vary according to the structural differences that may exist between one part of a deposit and another. The difference in the toughness values of a bed of shale and of a limestone lying over it in the same quarry is an example of variations due to differences in mineral composition and texture. Even when one bed is apparently similar to another lying over or under it their toughness values may vary. The following toughness values were obtained from blocks taken at vertical intervals of 10 feet from a dolomite deposit of fairly uniform character in a vertical direction. The deposit consisted of 32 feet of dark gray dense dolomite overlain by 18 feet of buff somewhat coarser dolomite (Plate XLII.). The toughness values were 13, 9, 10, 11 and 6. Jackson[14] found maximum differences of 9, 3 and 6 in a series of six samples taken from each of three limestone quarries.

The structural differences and consequent differences in toughness values found in broken stone fragments from the same deposit are in most cases due to minor planes of weakness that are scarcely if at all visible in the original deposit.

The determination of the location, direction and spacing of these minor planes of weakness is therefore important. In many cases the position and attitude of minor planes of weakness are related in a definite way to those of neighboring major planes. Lines of weakness are often present parallel to the main visible bedding planes, or there may be a tendency to part parallel to planes of foliation, flow cleavages; where there is a tendency to part along parallel incipient joints, fracture cleavage,[15] the lines of weakness are generally in two or more intersecting sets and are related to neighboring major planes or zones of fracture.

The presence of planes of weakness parallel to the bedding planes or planes of foliation is proved by the following experimental results. The results of tests by Clark[16] on Beekmantown

[14] Jackson, F. H., Jr., "Effect of Controllable Variables on the Toughness Test for Rock," *Proc. Amer. Soc. Test. Mater.*, Vol. XVII., Part II., 1917, pp. 571 to 585.

[15] Leith, C. K., "Structural Geology," pp. 61, 62.

[16] Reinecke, L., and Clark, K. A., *op. cit.*

dolomite from 6 localities, Chazy limestone from 27, Black River limestone from 7, and Trenton limestone from 23 localities gave an average difference of 2 in toughness values of cores drilled from the same block parallel and perpendicular to the bedding planes, those parallel to the bedding being the lower. In most of these tests there were no differences of structure visible to the naked eye in the cores themselves. Jackson[17] found that the average variation in toughness of cores drilled in directions parallel and perpendicular to the cleavage of a gneiss was 7 and of a schist 18. Sandstones from 3 localities drilled parallel and at right angles to bedding planes showed variations of 0, 3 and 4, respectively, limestones of 1, the lower values in all cases occurring in the cores parallel to the bedding or foliation planes.

In series of samples taken at three trap quarries, Jackson[18] also found that samples from the same quarry varied in toughness values up to maximum differences of 1, 6 and 7. In each quarry so tested the rock was apparently very uniform in lithological character and the variations must be due to planes of fracture cleavage. Fracture cleavage would also in part account for the differences in toughness values of the dolomites mentioned on page 571.

To arrive at the average toughness value of a deposit, therefore, it is necessary to determine the variations that exist in its mineral composition and texture, to determine if possible the structural variations within the deposit and to include as many of these variations as possible within the sample. The methods of sampling are discussed on page 566. Variation in toughness value may be due also to a slight variation of the diameter of the cores tested in the laboratory. The diameter of the cores should be kept within 1 millimeter of the standard.

The toughness values of over 90 per cent. of the road stones tested range from 3 to 25. An attempt to translate toughness values into the amount and character of impact that a stone can stand in the roadbed is given in the table on page 570. In the table under (*a*) maximum values are placed on the toughness of

[17] Jackson, F. H., Jr., *op. cit.*, p. 579.
[18] Jackson, F. H., Jr., *op. cit.*, p. 576.

stone used in waterbound macadam under light and moderate traffic because an excessively tough stone is usually resistant to abrasion, so that very light traffic does not grind off enough dust to keep the road firmly bound. Nevertheless the writer believes that it is the intention of the authors of these recommendations to emphasize the minimum rather than the maximum values, that it is, for instance, better to use stone of a toughness value of 20 than of 9 for moderately heavy traffic of say 200 vehicles per day.

Hardness Test.—The test for hardness is not of as great value as either the test of abrasion or toughness. Many engineers believe that it can be dispensed with. The work of Hubbard and Jackson[19] has proven that when the toughness value of a stone falls within the limits specified as suitable for macadam road construction under certain traffic conditions, the hardness value of the stone will fall within similar limits for hardness. The hardness value of a stone is, therefore ,indicated by its toughness value and the test for hardness can in most instances be omitted.

Cementation Test.—The cementing power of rock dust in a roadbed is due to the formation of colloidal materials in the hydrogel form, through the reaction of water on the fine rock particles. Cushman[20] has by chemical and microscopic work shown that when rock powders are ground with water in a ball mill a certain amount of hydrolysis takes place, giving rise in the case of most of the powders to the formation of alkaline hydroxyds and free silicic acid and the deposition of material of a gummy appearance on the outside of the fresh mineral particles. The gummy substance had the power of adsorbing the alkalies, in the solution. Dyes added to the solution were also adsorbed. By these and other tests he showed that these rock powders, after being acted on by water, had properties similar to those of coagulated colloids artificially prepared in the laboratory. These

[19] Hubbard, Prevost, and Jackson, F. H., Jr., "Relation between the Properties of Hardness and Toughness of Roadbuildings Rock," *Journal Agric. Reesarch,* Vol. V., No. 19.

[20] Cushman, Allerton S., "The Effect of Water on Rock Powders," U. S. Dept. Agric., Bureau Chemistry Bull. 92, Washington, 1905.

gummy glue-like films, which undoubtedly give rock powders their binding power, are, therefore, with reason, taken to be colloidal substances formed by the action of water on the powder. Continual wet grinding of the powder in the laboratory increases its binding power, because by exposing fresh surfaces of the rock particles to the water more of the glue-like substance is formed. Wet rolling of a roadbed has a similar result. The binding power and plasticity of clays are dependent in the same manner upon their colloidal content.

It has been assumed that the cementing power of road surfaces is due to the surface tension caused by hygroscopic or capillary water in the road surface. While such a bond must exist in a wet road surface, it must disappear when the road dries out. The surface of a sand road, for instance, is comparatively firm when wet but quite loose when dry. On the other hand a clay road containing much colloidal material becomes more coherent upon drying, and if some of the clay be burnt and all capillary and hygroscopic water driven out an even firmer bond is attained. While hygroscopic and capillary water may therefore aid in binding a moist road surface the main bond must be brought about by the action of colloidal material.

More than 95 per cent. of the road stones tested have cementing values lying between 10 and 200. Cementing values below 25 are considered low indicating stone not suited for waterbound macadam construction. Cementing values over 75 indicate stone that will bind readily into a firm mass. Results of tests made upon the same sample are liable to wide variation due to laboratory manipulation. As an example of possible variations of the results in testing, 5 briquettes made from the same batch of dough will break at values varying in many cases from 10 to 30 from the average value of the five. The average cementing values of a series of three samples made in three different laboratories gave the following values: Sample No. 1, cementing values, 28, 52, 63; No. 2, cementing values 18, 18, 25; and No. 3, cementing values, 27, 86, 75, respectively. The reasons for the variations are not fully understood although the fineness with which the "dough" is ground in the ball mill and total pressure

to which the briquette is subjected in the briquetting mold are probably two of the determining factors.

Absorption and Specific Gravity.—The amount of water absorbed by stone that is durable enough for macadam construction is small compared to that which passes into a waterbound macadam road surface by way of the interstices between the stones. And although in cold climates the freezing of the water in a road surface is one of the most potent factors in the disintegration of the road, the effect of the water absorbed by the stones, themselves, is probably very small. It is possible that the absorption of a stone may affect the strength of the bond formed by the stone with bitumen or Portland cement, but there are no definite data on that point. There is no direct relation between the specific gravity of a stone and its value on a road surface. It is used in computing the tonnage of stone present in a deposit from the measured cubic yardage.

Petrographic Work.

No attempt to discover the underlying reason for the fact that certain rocks make better roadstone than others, can proceed far without the aid of microscopic work. Its most useful result is that criteria can be derived that are recognizable with the naked eye or a hand lens, and that can be used for determining good roadstone in the field.[21] And because the rocks in one district may vary very greatly from those of the same composition and character in another, every one engaged in the study of road materials will experience the need of establishing his own criteria and continually revising them.

Comparative Value of the Different Classes of Rock.

The petrographic work done by Lord[22] upon the rocks tested in the laboratory of the Office of Public Roads furnishes a val-

[21] Reinecke, L., "Methods of Determining the Roadmaking Qualities of Deposits of Stone and Gravel," *Good Roads*, Vol. XIII., No. 7, May 19, 1917, pp. 294–297.

[22] Lord, E. C. E., "Relation of Mineral Composition and Rock Structure to the Physical Properties of Road Materials," U. S. Dept. Agriculture Bull. 348, Washington, 1916.

uable basis for the study of rocks from a road material standpoint. His work should be read in conjuction with a bulletin prepared by Hubbard and Jackson[23] in which the results of physical tests upon some 3,650 samples of rock together with their petrographic names are given. The great variation in the values of samples of the same class of rock is demonstrated in a table accompanying the latter bulletin.

While it is necessary to emphasize the large diffences that are found in any one kind of rock, the results show that certain rock classes yield better road stone on the average than others. It has been established that in general the igneous rocks are more durable than their foliated metamorphic equivalents, the gneisses and schists, and than the sedimentaries. Among the sedimentary rocks limestones and dolomites form the principal road builders, only certain varieties of sandstone can be used, and the shales as a class are not considered good for road construction. The same is true of the metamorphosed sedimentary rocks excepting quartzite. From the results published in an earlier bulletin[24] of the same laboratory the writer has attempted to compare the relative toughness of certain of the more important groups of road building rocks by calculating the percentage of total samples tested whose toughness values fell within the limits assigned for light, moderate and heavy traffic[25] respectively. They may be grouped according to toughness value in the following order beginning from the highest: traps, felsites, gabbroic rocks, granitic rocks and limestones. For the purposes of this grouping the traps include basic hypabyssal and extrusive igneous rocks, andesite, basalt, diabase, etc.; felsites include acid representatives of the same types, rhyolites, quartz-porphyries, etc.; gabbroic rocks include basic plutonic varieties, diorite, gabbro, etc.; and granite rocks include the acid representatives of the plutonic rocks, granites, syenites, etc.; and limestones include both lime-

[23] Hubbard, Prevost and Jackson, F. H., Jr., "The Results of Physical Tests of Road Building Rock," U. S. Dept. Agric. Bull. 370, Washington, 1916.

[24] Goldbeck, A. Y., and Jackson, F. H., Jr., "The Physical Testing of Rock for Roadbuilding," U. S. Dept. Agric., Office of Public Roads, Bull. No. 44, Washington, D. C., 1912.

[25] Geol. Surv. Canada Mem. 85, p. 26, Ottawa, 1916.

stones and dolomites. A grouping[26] according to per cent. of wear made in the same way would be as follows, beginning with those having the lowest per cent. of wear (greatest resistance to abrasion): Diabase and basalt, diorite and gabbro, andesite and rhyolite, granite, limestone, dolomite.

From the results given by Lord[27] it appears that the average cementing values of the more basic igneous rocks of the plutonic type are higher than those of the acid, and the cementing value of the extrusives higher than that of the coarser grained plutonic rocks.

The same author calls attention to the relationship that exists between the road making value of the rocks and their mineral composition, texture and amount of alteration. In igneous rocks of the plutonic type those high in hornblende and augite tend to be more durable, those high in mica less durable than the average. Fine-grained holocrystalline rocks are apt to be more durable than coarser-grained varieties. Secondary alteration to epidote and uralitic hornblende tends to increase; and to kaolin and chlorite, etc., to decrease the durability of the plutonic rocks. The cementing value of the holocrystalline (plutonic) rocks varies inversely as their quartz content and directly according to the amount of secondary alteration. The latter increase is more noticeable in the case of the less quartzose varieties, syenite, gabbro, etc. In igneous rocks of the extrusive and hypabyssal types relation between mineral composition and durability is indicated by the lower percentage of wear, greater durability, of the diabases, basalts and trachytes with little or no glass, and the rhyolites where glass is present. The high toughness of trachyte, diabase, and basalt compared to andesite, is due to the textures of the former three types in which fine crystalline constituents interpenetrate and interlock; and this would also account for the increase in toughness observed in certain diabases with much secondary uralitic hornblende. Porphyritic and glassy textures may, however, also show very high toughness values. Among

[26] This is based on the diagram on page 4 of Bull. 370, U. S. Dept. of Agriculture, Washington, 1916.

[27] *Op. cit.*, Table 1, page 2.

the rocks studied by the writer those showing the highest toughness values were a quartz syenite prophyry with large phenocrysts in a very fine crypto-crystalline groundmass in which the constituents had a feltlike interlocking texture. Fineness of grain with consequent lack of the planes of weakness due to large mineral cleavage planes accounts for the higher toughness observed in the extrusive and hypabyssal as opposed to the plutonic rocks. Secondary alteration tends to weaken the finer grained igneous rocks excepting where accompanied by the formation of epidotes, zeolites and uralitic hornblendes, which appears to increase their general durability.

Although the gneisses are supposed to be less durable than their unmetamorphosed counterparts, the writer encountered many deposits of igneous gneiss[28] in a certain district that gave excellent results for wear and toughness in the laboratory and from some of which very satisfactory road surfaces were constructed. In this area there was a well-defined distinction between the results of tests on gneisses with or without augite, and tests indicated that the rocks carrying augite were of greater durability although both series had been subjected to about the same degree of metamorphism. The general observations upon the influence of mineral composition and secondary alteration upon the road-building values of igneous rocks holds as far as the writer's experience goes also for the gneisses of igneous origin, but, although one would expect the less foliated and finer grained gneisses to be more durable than coarser grained or more intensely foliated ones of the same composition, that is not in every case true. The gneisses of sedimentary origin are as a rule unsuited for road work. Of the schists those carrying large percentages of hornblende give better results in the laboratory tests than chlorite or mica schists. This is due in large part to the arrangement of the cleavages of the mica crystals in the mica schists which lie in parallel planes allowing the stone to split easily along these planes, while the cleavages in the hornblendes of a hornblende

[28] Geol. Survey Can., Memoir 99, pp. 142, 143. Ottawa, 1917, and later work, unpublished.

schist, although parallel to one axis, lie in planes that are inclined to each other.

Of the sedimentary rocks the limestones and dolomites are by far the most useful for road construction. They are of only moderate durability and not as a rule suited for roads subjected to heavy traffic except when used with bituminous, Portland cement or other equally efficient artificial binders, with which they appear to have given as satisfactory results in some cases as the more durable igneous rocks.[29]

The durability of limestones and dolomites varies to some extent according to texture, the finer and more even grained varieties being the more durable.[30] Most of the limestones cement together with ease in a roadbed, more so than their average cementing values, as found in the laboratory, would indicate. The rapidity with which the surfaces of certain limestone roads will change after a rain, from a dusty condition into one in which the dust has become practically incorporated back into the road, seems to indicate that in addition to any colloidal action that may have taken place, actual solution and re-cementation has had a part in binding the surface. This is probably not true of the other less soluble roadmaking rocks.

Sandstones are not used as extensively as the limestones for road work. Their strength and cementing value are probably largely functions of the strength and amount of the cement that binds their grains together, although the shape of the grains also plays a part. In the laboratory, their cementing values are as a rule low and the results obtained in the roadbeds are, moreover, in many cases not as good as the laboratory results would indicate. Certain argillaceous, calcareous and ferruginous sandstones have made satisfactory waterbound macadam roads, but the majority of sandstone deposits are unsuited for roadwork. Limestones and sandstones impregnated with bitumen, the so-

[29] Reeve, Chas. S., and Lewis, Richard W., "Toughness of Bituminous Aggregates," *Journal Agric. Research*, Vol. X., No. 7, Aug., 1917, pp. 325–326; Nash, J. P., Proc. Amer. Soc. Test. Mat., vol. XVII., 1917, Part II., pp. 411, 414.

[30] Geol. Survey of Canada, Memoir 99, pp. 18 and 179.

called rock asphalts, are in a class by themselves. They are of very rare occurrence, but in places are very valuable sources of paving material. The asphalt pavements in European cities are built largely of rock asphalt, limestones impregnated with bitumen, from Italy and Southern France,[31] that is, these rock asphalts are used in the construction of one of the very best grades of road surface.

Rock asphalts occur also in Kentucky and Oklahoma and some country roads have been constructed with them. The virtue of the rock asphalts lies of course in their bitumen content rather than in the stone as such.

Shales are as a class unsuited for road construction.

The metamorphosed sedimentary rocks, marble, quartzite, slate, etc., are not in general suited for road aggregates except the quartzites, which are fairly durable though lacking in cementing value. Of the rocks formed practically entirely as precipitates from solution, the cherts have been used with success in road building.

GLACIAL BOULDER DEPOSITS.

Glacial boulders are widely scattered over the northern sections of the United States and Canada. They are found piled in fences and heaps in the fields, and these piles form a cheap and often valuable source of roadstone. In the production of crushed stone from boulders the crusher can be located next the proposed highway, the boulders hauled there in winter and put in the crusher with little or no preliminary breaking, eliminating quarrying expenses.

The disadvantages of using this form of material for road surfacing arise from its heterogeneous character, which may cause one section of a road to wear more rapidly than another or may give rise to uneven wear in the same section of road. The boulder deposits are aggregates of many varieties of stone, each of which may vary in roadbuilding character from its fence neighbor, and the proportions of these varieties in any one fence

[31] Durham, H. W., "Street Paving and Maintenance in European Cities," p. 203, published by the Borough of Manhattan, New York City, 1913.

may, and do in many cases, vary decidedly from that in an adjacent fence or from one farm to the other.

During the last three years the Geological Survey of Canada has had occasion to investigate the boulder deposits along several hundred miles of proposed highway including many thousands of fences. The field methods used were as follows: Fences and piles were plotted on a fairly large scale topographic or township plan map, at least 1 inch = 1 mile, the quantity calculated, and estimates made of the proportions of boulders under and over 1 foot in diameter. The composition was estimated by counting the boulders of each variety present in a section of fence and such estimates were made in practically every fence. The true average composition of a group of fences was obtained from the composition of the individual fences, by giving due weight to the yardage included with each individual estimate.

The reason for estimating the amount of stone under one foot is that boulders of that size can be handled by one man, and are about the largest size that will go into the average portable crusher without preliminary breaking. Since fences generally follow or lie parallel to land lines they can be plotted very rapidly and the estimates of quantity and composition can be made with very fair accuracy by comparatively unskilled assistants. The interpretation of results is not however an easy matter. One realizes that the quality of the aggregate must vary according to the variation in the quality and proportions of its component boulders. The aggregates are made up of from, say, four to ten different kinds of stone, each of which may vary in quality from the other and the proportions of these variables are also apt to vary so that a sample tested from one fence does not give a clue to the character of a sample from another. After several attempts at a solution the following system was adopted with satisfactory results.

It is a fact well known to geologists that glacial deposits, with some exceptions, carry a high percentage of boulders derived from the bedrock deposits in which they lie. In those portions of the Ottawa-St. Lawrence valleys covered by our surveys, the deposits in any one area can be grouped into two or three series.

If the area lies over the Trenton formation, for instance, a large proportion of the stone will be Trenton limestone, another portion may be Beekmantown dolomite while a third may consist of say Pre-Cambrian gneisses. In another area not far away Potsdam sandstone may be combined with Beekmantown dolomite and the gneisses. It was found by sampling each of these series separately that the quality of any one series was fairly constant over an area of a number of square miles. In areas where there are large percentages of Pre-Cambrian schists and gneisses it is often possible to subdivide the gneisses into two series. Composition estimates consisted in determining the proportions of these series present in the deposits. It is essential only that the rocks forming a series be of about the same durability and one series be easily distinguished from another in the field. This system was tried only after other schemes had proven unsatisfactory. Knowing the composition of the aggregates their durability or resistance to abrasion was arrived at in the following manner: Experiments conducted by Clark[32] established the fact that the per cent. of wear of any combination of these series could be calculated from the per cent. of wear of each series and their relative proportions in the combinations. If the per cents. of wear of the various series are expressed by W_1, $W_2, \cdots W_n$ and the percentage proportions in which they occur in the mixture by C_1, $C_2, \cdots C_n$—the per cent. wear of the mixture W_m is given by the formula $W_m = \Sigma CW/100$.

In other words the results of estimates of composition based on these natural rock groupings can be translated directly into figures indicative of the durability of the aggregates. One set of samples and tests suffice for the whole area occupied by combinations of the same major series. That is, if Trenton limestone forms 40 to 90 per cent. of the stone over an area of 30 square miles and it is mixed with either or both Beekmantown dolomite and Pre-Cambrian gneiss within that area, one set of tests for per cent. of wear of each of the three series will enable

[32] "The Sampling of Deposits of Roadstone and Gravel in the Field," by L. Reinecke and K. A. Clark, *Proc. Amer. Soc. Testing Materials*, Vol. XVIII., presented at the annual meeting, June, 1918.

the investigator to determine variations in quality of the stone from one part of the area to another, and the average composition of any given quantity of stone will also enable him to calculate its average durability.

The other tests made upon boulder aggregates are those for cementing value and specific gravity. Tests for toughness and hardness are not practicable because of the large number required. Because of the difficulty in getting closely concordant results on tests for cementing value the method outlined above was not applied to that test. The test for specific gravity is made as in that for the coarse aggregate in the gravel tests.

In observations made upon broken stone derived from boulders under actual service conditions in the roads, it was found that because of the weathering they had undergone the durability of boulders of certain of the sediments, limestones and especially impure dolomites, were much lower than bedrock deposits of the same character. The cementing values of the igneous gneisses were uniformly good, and combinations of boulders like quartzitic Potsdam sandstone, of poor cementing quality, with boulders of igneous gneisses, bound fairly well in the roads even with aggregates carrying as much as 60 per cent. of sandstone.

GRAVEL DEPOSITS.

From the standpoint of the testing engineer, gravel is that portion of a natural mineral or rock aggregate consisting of particles over ¼ of an inch in diameter, while sand is the portion consisting of the particles of diameter smaller than ¼ inch. Natural deposits of gravel invariably contain sand, sometimes in much greater proportion than the material of gravel size. In the following section natural deposits of gravel and sand are referred to as gravel. Natural deposits of sand alone were not investigated because there was no demand in the districts surveyed for such work. Because of a lack of experience with the investigation of material for sand clay and topsoil roads also, those two important forms of road surfacing are not referred to in this discussion.

Field Methods.

In field work upon gravels the writer found it convenient to work to the following schedule:

Character of Gravels.	*Exploitation of Gravels.*
Map location.	Owner's name and address.
Texture.	Amount excavated and available.
Impurities.	Uses and results in service.
Composition.	Prices of gravel.
Shape of pebbles.	Possibilities of production.
Geological observations.	Costs of land, of gravel production.
	Transportation facilities.

The boundaries of gravel areas were sketched largely from surface indications, supplemented by all the data obtainable from pits, wells, and borings. Where land values are high or gravel is very scarce it sometimes pays to sink holes to test the extension of the gravel. A posthole auger with four- to six-inch bit, a movable cross bar and one or two extra rod lengths which can be clamped together, can be used by one man to dig holes 10 to 12 feet deep in gravel that is not too coarse.

For studying gravel textures or grading a set of screens and a spring scale of the type used by fishermen are needed. The ordinary round laboratory screen with tight fitting cover and bottom would give the most accurate results, because the gravel can be given a more thorough shaking than when the screen is uncovered. They are bulky, however, and therefore difficult to carry. The writer has used a set of screens of the shape of cylinders divided in two longitudinally. These were made to rest one in the other and all of them fitted into a sheet-iron cylinder with cover. The screens themselves had no covers and the screening was a lengthy process. Proportions were estimated by volumetric measurements. The actual percentages of gravel passing one of these screens and retained on the next varied by from 5 to 10 per cent. from laboratory analyses of the same gravels.

The principal impurities to be looked for are clay, iron oxide, lime and alkali crusts, and organic matter. A rough estimate of the amount of clay present can be made by thoroughly shaking

up a sample of the gravel with water in an ordinary fruit jar and allowing it to settle. The clay and silt settle in a thin layer on top of the gravel and the thickness of the clay layer in relation to that of the whole sample can be taken as the proportion of clay present. An idea of the amount of organic material present may be obtained by digesting some of the sand in sodium hydroxide according to the method proposed by Abrams and Harder, page 591. There is no ready way of determining the amount of lime, alkali and iron crust present quantitatively.

A knowledge of the pebble composition of the gravels is desirable because they give a clue to the durability of the material. In order to be of use in comparing one gravel with the other they should all be made on pebbles lying between the same size limits. The pebbles between ½ and 3 inches are of a convenient size for such an analysis. It has been found that, in some gravels at least, the percentage composition of the larger pebbles differs radically from those of the smaller. The shape of the pebbles are supposed to affect the readiness with which they bind into a mass in a road bed. Geological observations that tend to throw light on the origin of the gravel are useful in that they help to determine the shape and extent of the deposits.

That part of the schedule dealing with the exploitation of the gravels is self explanatory.

Laboratory Tests.

The tests made on gravels and sands are those to determine their texture, percentage of voids, specific gravity, resistance to abrasion and cementing value. Sands and gravels used in concrete work are further subjected to mortar tests and tests to determine the amount of organic impurities they contain.

Analyses to Determine Texture.—The texture is determined by mechanical analyses in which the material is separated into a number of sizes by shaking through a series of screens or sieves. The size of the openings in each screen or sieve are equal and diminish progressively from screen to screen through the series. The standard method of screening and the size of openings in

the screens that should be used for making a mechanical analysis of broken stone or broken slag are specified by the American Society of Testing Materials.[33] This is applicable also to gravel. The material passing the ¼-inch screen is called the "fine aggregate" or sand and is analyzed separately. The standard[34] method for making a mechanical analysis of fine aggregate except that for use in cement concrete is described in the same volume, pp. 535, 536. In the Tyler system of sieves (Plate XL., *B, b*), the length of one side of the square openings in any sieve is approximately four times the length of the side of the square openings in the next smaller sieve, throughout the series. That is, the proportion between the areas of the openings in any two adjacent sieves in the series is the same throughout the series. It follows that by means of these sieves the material is separated into a series of lots of particles of different sizes, in which the average size of grain in any one lot has a definite and constant proportion to the average size of grain in the next lot. The openings in the standard series of sieves and screens are in no such definite proportions to each other and the Tyler system seems to be the more logical one to use. Analyses made by the Tyler system can be conveniently converted to that of the standard by plotting the cumulative or total percentages retained on each screen in a diagram (Fig. 57) and reading off the points where the texture curve crosses the ordinates of the standard screens.

Voids Test.—The voids in the gravel or sand may be determined by the cone specific gravity method.[35] In this test a truncated steel cone (Plate XL., *B, c*), 10 inches in over-all height, 10 inches in over-all diameter of bottom, and 3 inches inside diameter of opening is filled with the gravel, which is thoroughly compacted by oscillation and kept full until no more gravel can enter. Then if $A =$ the weight in grams of the cone empty, $B =$ weight

[33] *Amer. Soc. Test. Mater.*, A. S. T. M. Standards, issued 1916, p. 537, Philadelphia, Pa.

[34] *Ibid.*, pp. 535, 536.

[35] Blanchard, Arthur A., "Elements of Highway Engineering," pp. 494, 495. John Wiley & Sons, New York, 1915.

in grams of the cone filled with clean water, C = weight in grams of the cone filled with compacted aggregate, D = specific gravity of the aggregate the

$$\text{Percentage of voids} = \left(1 - \frac{(C - A)}{(B - A)D}\,100\right).$$

Specific Gravity.—Specific gravity determinations on non-homogeneous mineral aggregates like gravel are made separately

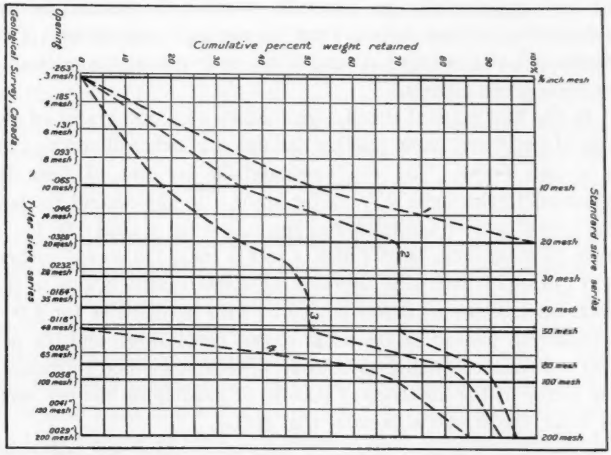


FIG. 57. To illustrate the variation of the compressive strength of sand cement mortars according to variations in the coarseness and texture (grading) of the sands. (Based on the results of experiments by F. H. Jackson, Jr.)

on the material over ½-inch and under ½-inch in size. An "apparent" specific gravity determination is made of the coarse and a true specific gravity determination of the fine aggregate. Knowing the proportions of coarse to fine aggregate present an approximate apparent specific gravity can be calculated for the whole.

The recommended method of determining the apparent specific

gravity of coarse aggregates[36] is described in the *Proceedings* of the American Society for Testing Materials.

The more satisfactory methods of determining the specific gravity of coarse and also of fine aggregates are described and compared with illustrations of the apparatus used in a paper by Hubbard and Jackson.[37]

Abrasion.—The tests on the abrasion of gravels are made in the Deval abrasion machine, but as the gravel particles are not of the same size and are generally rounded the results are not comparable to those derived from the test on broken stone. Two methods have been used in our laboratory in testing the resistance to abrasion in gravels.

In the first method the charge consists of 5,000 grams of the run of the gravel bank, passing through a 2-inch and retained on a ½-inch screen. The test is run and the per cent. of wear determined in the same way as for stone. In the second method the gravel is first screened into the following 5 sizes: ¼ to ½ inch, ½ to ¾ inch, ¾ to 1 inch, 1 to 1¼ inch, 1¼ to 1½ inches. 500 grams of each size aggregating 2,500 grams is run in the abrasion machine with 20 steel balls 1 inch in diameter for 5,000 revolutions, instead of the usual 10,000 revolutions, and the per cent. of wear calculated as before. The first method introduces the factor of the influence of texture or grading on the per cent. of wear, the second eliminates that factor.

Cementing Value.—Tests for cementing value are made by taking 500 grams of the fine aggregate and the proceeding is the same as in the case of stone.

Mortar Tests.—Sand, that is the material under ¼ inch, which is to be used in cement concrete construction whether of roads or other structures is tested in the form of mortars made of the sand and Portland cement, in comparison with mortars made of a standard sand and the same cement. The mortars are moulded into cylinders and briquettes of standard size and after seasoning are tested for compressive and tensile strength.

[36] *Proc. Amer. Soc. Test. Mat.*, Vol. XVII., 1917, Part I., pp. 776, 777.

[37] Hubbard, Prevost, and Jackson, Frank A., Jr., "The Specific Gravity of Non-homogeneous Aggregates," *Proc. Amer. Soc. Test. Mat.*, Vol. XVI, 1916, Part II., pp. 378–402.

The detailed methods for making these comparative tests are the same as those followed in testing Portland cement by means of mortars made with standard sand. These methods are described with diagrams of the moulds to be used.[38] Great care in manipulation is needed in these tests and the methods as described by the Society should be followed in detail to obtain results of any value.

Tension tests have been made by Nash[39] on briquettes of concrete of the same shape as those used in the mortar tests.

Test of Organic Impurities.—A method for testing the organic impurities present in sands has been developed by Abrams and Harder.[40] In brief the test consists of digesting a sample of sand in a solution of sodium hydroxide. If the sand contains organic impurities, thought to be largely of a humus nature, the filtered solution resulting from this treatment will be found of a color ranging from light yellow through red to black. The depth of color which is measured by comparison with a series of solutions of alkaline sodium tannate, has been found to furnish a measure of the effect of the impurities on the strength of mortars made from such sands. For the details of laboratory manipulation the reader is referred to the circular just quoted. For field use, fill a graduated 12-oz. prescription bottle to the 4½ oz. mark with sand, add a 3 per cent. solution of sodium hydroxide until the volume of sand and solution after shaking amounts to 7 ozs. Shake thoroughly and allow to stand over night. If the clear supernatant liquid is colorless or light yellow the sand is satisfactory as far as organic impurities are concerned. If the color is dark red to black the sand should be subjected to the standard mortar tests before being used in concrete work.

[38] For tension strength tests, see *Am. Soc. Test. Mat.*, Standards issued 1916, pp. 446–448. For compression tests see *Proc. Am. Soc. Test. Mat.*, Vol. XVI., 1916, Part I, pp. 590 to 593.

[39] Nash, J. P., "Tests of Concrete Road Aggregates," *Proc. Amer. Soc. Test. Mat.*, Vol. XVII., 1917, Part II., pp. 396–421, with diagrams of molds and apparatus for tension strength test.

Value of the Tests.

The average composition of the smaller particles in a body of gravel or sand may vary from that of the larger particles, and the texture or grading, that is, the relative amounts of the different sizes present in any part of a deposit, may, and usually does, vary from the texture in other parts (Plate XLIII., *A*). It is therefore difficult to determine the average physical properties of gravels and sands or their probable behavior in roads and other structures by the method of sampling and testing.

Analyses to Determine Texture.—The value of a gravel or sand in structural work is governed by its texture or grading in more cases perhaps than any other one of its physical characteristics. The larger pebbles in gravel macadam do not wear as fast as the smaller and the largest pebbles will finally project through the top and cause a rough surface. The density and resilience of the surface course is assumed, too, with good reason, to be governed very largely by the balance between the larger and smaller sizes of the gravel, the best combination being one in which the larger stones are not too far apart and the voids between them have been filled to the greatest possible extent by small particles. A lack of fine material prevents rapid binding of the surface and too much fine sand relative to the coarser gravel sizes make a loose sandy road surface which is apt to wear to dust fast.

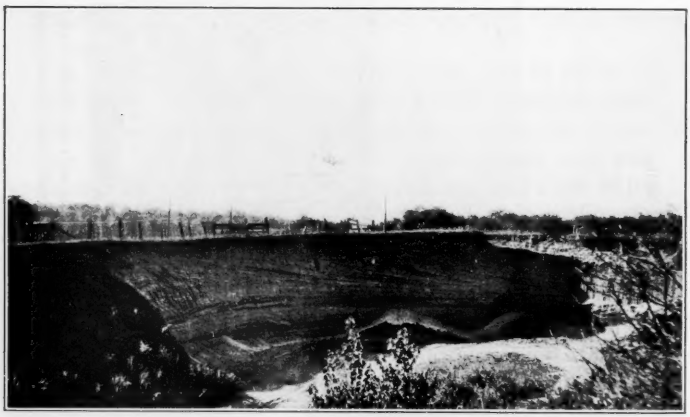
In cement concrete work it has been proven, both in the case of sand in sand mortars and of gravel and sand in concrete aggregates, that there is an increase in strength of the mortar or concrete with an increase in coarser sizes up to a certain point without much regard to the grading. Beyond that, with a lack of fine material in the aggregate, that is, with very poor grading, the strength falls.

In Fig. 57, for instance, the texture curves of well-graded sands, that is, those in which the proportions of the sizes retained on various sieves are the same, would cross the diagram from left to right in nearly straight lines, and as the proportion of coarser sizes increased, the curves would move upward and to the left.

Fifty-foot wall of dolomite, with quarry of the Canada Crushed Stone Corporation at Dundas, Ontario. To illustrate uniform results from abrasion and varying results from toughness tests on rock of uniform lithological character.

A.

Lense of gravel overlain by silt in an esker at Trenton, Ontario. On the right, on the upper side of the lense, the sharp change horizontally from gravel to silt is illustrated.



B.

Gravel pit at Cedar Springs, Ontario, illustrating variations in texture within a body of gravel. The dark streaks are beds of silt; the rest of the wall is of sand and gravel of varying fineness.

Jackson's[41] work on the relative strength of sands of varying texture, illustrated in Fig. 57, shows that the compressive strength of the mortars increase as the curves move to the left and upward, that is, as the coarse sizes increase, up to the maximum, curve, No. 2, which lies above and to the left of the center of the diagram, but this curve does not indicate a smoothly graded sand. The coarser sands whose curves lie still further to the left, however, give lower compression results. Sands whose curves lie within the zone between curves 1 and 3 have within 40 per cent. of the maximum compressive strength irrespective of their smoothness of grading. The texture curves of the sixty texture combinations tested by Jackson lie within the zone bounded by curves 1 and 4 in the diagram. Abrams[42] has proven the same principle for aggregates consisting of both gravel and sand, that is for concrete mixtures.

The effect of the texture of the aggregate on the quality of concrete and other forms of road construction being established, the need of discovering the texture of a natural gravel deposit or a gravel stock pile is clear.

The value of the test however depends not only upon the proper interpretation of the results, but in just how far those results represent the average texture of the body of gravel that is being tested. The writer's experience has been that variations may arise through laboratory manipulation and sampling. There may be differences of texture ranging up to 5 per cent. of the amount retained on any one screen, between two portions taken for analysis from the same 40-lb. laboratory sample bag, and field sampling must be done with great care if the sample is to represent the small section of gravel surrounding the point sampled.

[40] Abrams, Duff A., and Harder, Oscar E., "Colorimetric Tests for Organic Impurities of Sands," Circular No. 1, Structural Materials Research Laboratory, Lewis Institute, Chicago, 1917; also *Proc. Amer. Soc. Test. Mat.*, Vol. XVII., 1917, Part I., pp. 327–333.

[41] Jackson, F. H., Jr., "Influence of Grading in the Value of Fine Aggregate used in Portland Cement Concrete Construction," *Journ. Agricultural Research*, Vol. X., No. 5, July 30, 1917, pp. 264–274.

[42] Personal communication from Professor D. A. Abrams, of the Lewis Institute, Chicago.

Great differences in texture are moreover to be looked for within the same natural gravel deposit (Plate XLIII., *B*).

In the following tables recommendations for the textures of sand and gravel to be used in gravel macadam and in sheet asphalt are given. It will be seen that in each case a certain amount of latitude is given for variations in the texture. Until sampling and testing can be done with greater efficiency a certain amount of latitude will always be necessary.

SPECIFICATION FOR GRAVEL FOR WATERBOUND GRAVEL ROADS.
Cumulative Percentages Retained on Screens.

	3 In.	2½ In.	2 In.	1½ In.	1 In.	¾ In.	½ In.	¼ In.	Passing 200 Mesh.
1. Waterbound macadam wearing course.....				0		15 to 55		60 to 75	5 to 15 A
Do.....			0 to 5		10 to 55			50 to 75	5 to 15 B
2. Base course....		0			15 to 55			60 to 75	5 to 15 A
Do.....	0 to 5				10 to 55			50 to 75	5 to 15 B

A. Recommended by the Amer. Soc. of Civil Engineers, January, 1917.
B. Recommended by the Office of Public Roads, Washington, D. C. Percentages to the nearest 5 per cent.

RECOMMENDED[43] SPECIFICATIONS FOR SAND FOR SHEET ASPHALT WEARING COURSE.

	Per Cent.
Passing 10-mesh sieve.....	100
Total passing 10-mesh sieve and retained on 40-mesh sieve.....	12 to 50
Passing 10-mesh sieve and retained on 20-mesh sieve.....	2 to 15
Passing 20-mesh sieve and retained on 30-mesh sieve.....	5 to 15
Passing 30-mesh sieve and retained on 40-mesh sieve.....	5 to 25
Passing 40-mesh sieve and retained on 50-mesh sieve.....	5 to 30
Passing 50-mesh sieve and retained on 80-mesh sieve.....	5 to 40
Total passing 80-mesh sieve and retained on 200-mesh sieve.....	20 to 40
Passing 80-mesh sieve and retained on 100-mesh sieve.....	6 to 20
Passing 100-mesh sieve and retained on 200-mesh sieve.....	10 to 25
Passing 200-mesh sieve.....	0 to 5

Voids Test.—The voids in a sand or gravel are related to the grading of the sand, to the arrangement of the particles and to

[43] These specifications are those recommended by the Office of Public Roads and are practically identical with those recommended by the American Society of Civil Engineers.

its moisture content. If the moisture content and arrangement of the particles are the same in the surface of a gravel macadam road, as in the gravel under the conditions of the void test, it may be assumed that the gravels showing the lowest percentage of voids in the laboratory will give the densest mixture and therefore the one that is the most easily bonded in the roadbed. Since the conditions of moisture and arrangement of particles probably vary greatly under road conditions from those in the laboratory there is no reason for believing that the percentage of voids in the laboratory is an indication of the voids to be expected in the roadbed.

Abrasion Test.—The abrasion tests for gravel are just now in the experimental stage. Several methods are being tried out at different laboratories. It is probable that there will be an opportunity for a thorough comparison of these methods and the evolving of a satisfactory standard method within the next few years.

Cementation Test.—The principles of cementation in a gravel macadam road and in the laboratory tests are the same as that for broken stone.

Mortar Tests.—The compressive and tensile strengths of cement sand mortars relative to the strength of a mortar made of standard sand and cement under the same conditions has been the generally accepted method for determining the value of the sand for concrete work. During the last few years many engineers have become dissatisfied with the tests both because of the great possible variation in results of tests on the same sands, and because the test of the sand mortars, or the fine part of an aggregate mixed with cement, were not believed to be a fair test of the behavior of the whole aggregate mixed with cement in the form of concrete. The proportioning of the sand and cement mortars by weight in the laboratory, while volumetric proportions are used in actual construction, has also been subjected to criticism. Abrams[44] has shown by experiment that the compressive strength of concrete is a function of the proportions of the volume of

[44] Abrams, D. A., "The Basic Principles of Concrete Mixtures," *The Concrete Highway Magazine*, Vol. 2, No. 5, May, 1918, pp. 110 to 113.

water to the volume of cement in the mixture, and it is certain that the variable amount of water used in the mortar tests, gives rise to differences in strength irrespective of the value of the sand.

The question of the value of mortar tests and of the proper means of arriving at the most efficient concrete mixtures, have been the subject of much study and experimentation within the last few years and these problems are by no means solved. The proceedings of the American Society for Testing Materials for the years 1916, 1917 furnish with other publications a number of interesting papers bearing on these subjects.

SUMMARY.

The paper is intended as an introduction to the study of the natural non-bituminous road materials, that is, stone and gravel.

Methods of Investigation.

It is believed that detailed examinations of the materials needed for certain roads or occurring within certain areas are of greater practical importance than general surveys covering larger areas. The materials are examined and mapped in the field and roads made from them are inspected. Samples taken from certain deposits are tested in the laboratory and petrographic studies are made of the tested material.

Bedrock Deposits.

In field work on outcrops of bedrock it is good practice to classify the material into geological series or formations first, with further subdivision into lithological units where necessary. Particular attention should be paid the systems of fracture and foliation planes in the deposits in order to obtain clues to the arrangement and number of the obscure planes of fracture and flow cleavage that accompany them and which may seriously affect the uniformity of the road-making value of a deposit.

In attempting to judge of the value of either stone or gravel in a road surface it is necessary to know, and recognize the influ-

ence of, the other factors that determine the condition and serviceability of a roadbed. They are the volume and intensity of the traffic, the strength of the foundation upon which the surface rests, the manner in which the surface itself has been constructed, and the effectiveness with which water is drained away both from the surface and the base of the road.

The laboratory tests made on road stone are those to determine resistance to abrasion, toughness, hardness, cementing value, specific gravity and absorption. The taking of samples for laboratory tests should be made with care so as to include as many of the variations within the deposit as possible.

The per cent. of wear of the sample tested in the abrasion machine is a satisfactory measure of the probable resistance to abraison of the stone from a deposit under traffic conditions. The toughness value of a laboratory sample, on the other hand, represents the resistance to impact of only one part of the deposit tested. It may vary considerably from the average for the whole.

The hardness test can probably with advantage be dispensed with, and that for absorption is not of very great importance.

The cementing power of stone in a road surface is due to colloidal material developed through the reaction of rock dust with water. The same material is produced during the grinding of the rock powder with water in the cementation test. Unfortunately it is still difficult to obtain closely concordant results from the laboratory tests.

Microscopic examinations of the rocks tested in the laboratory is useful in establishing criteria for use in the field.

The road-making values of the rocks of the same class vary greatly but certain classes are on the average more durable and have better cementing values than others. The texture, mineral composition, and the degree and character of the secondary alteration to which they have been subjected, all influence their road-making properties. In a very broad classification of rocks according to their road-making values basic igneous lavas and dike rocks would come first. Acid igneous rocks of the same

types are nearly equivalent in their road-making properties and the coarser grained holocrystalline rocks are of lesser values. Limestones and dolomites are of intermediate value, only certain of the sandstones and practically none of the shales are of use in road building. Of the metamorphic rocks certain igneous gneisses and hornblende schists have made good roads but in the majority of cases the metamorphic rocks are not as good road makers as their unmetamorphosed representatives.

Glacial Boulder Deposits.

In investigating boulder deposits, it has been found that their average composition, expressed in the percentages of a few main rock types present in the boulder mixtures can be determined rapidly in the field. The main types of stone which go to make up the mixtures are sampled separately and tested in the abrasion machine. By a formula derived experimentally the resistance to abrasion of any mixture as found in the field can then be calculated.

Gravel Deposits.

Gravel deposits are tested to determine their grading, percentage of voids, specific gravity; resistance to abrasion and cementing value. Sands and gravels, that are to be used in cement concrete work, are tested in the form of mortars or concrete made from them with Portland cement. Tests are also made to determine the amount of organic impurities present in concrete sands or gravels.

Analyses to determine texture or grading are important, since the value of a gravel or sand for road construction depends greatly on its texture. It has been established also that a rather coarse sand makes a stronger mortar with Portland cement than a very fine one; and that a very poorly graded sand is liable to yield a weak Portland cement mortar. The usefulness of the test for voids is a matter of debate. The tests for abrasion and cementing value are of use in judging of the quality of a gravel for waterbound macadam purposes. Mortar tests are the ac-

cepted method of testing the quality of sand for cement concrete purposes. Great care is necessary in the performance of these tests, and many factors, other than the actual structural value of the sand, may affect the results of the laboratory tests. The test for organic impurities has not been in use for a very long time, but promises to be of great value.

ANOMALOUS DIPS.

EUGENE WESLEY SHAW.

INTRODUCTION.

In connection with Dr. Mather's[1] recent paper in ECONOMIC GEOLOGY I should like to call attention to some additional examples of peculiar dips and to make some remarks concerning the causes of dips.

It would be very convenient if we had a suitable class term to designate those special aberrant dips of single strata or small groups of strata that are independent of the general structure at any locality. Mather's expression "superficial dip" is not quite satisfactory, one reason being that many of these dips are original and altogether independent of the present surface. "Local dip" has come to imply lack of harmony with the general dip in a region, rather than a special dip of some stratum compared with other strata in the same locality. "Anomalous dip" seems at present the most satisfactory designation. This may be shortened if need be to "anomocline," contrasting with homocline—sameness or uniformity of dip, though homocline generally implies areal uniformity. Anomalous is said to come from the Greek words "*an*" (without) and "*homos*" (sameness). Apparently the word might as well have been derived from the Greek "*a-nomos*" (lawless) and still be as useful for present purposes.

Petroleum geologists are continually on their guard for these "anomalous" dips. Everyone of them has had to deal with many such dips and it is safe to say that most have at one time or another been misled because of the lack of parallelism among strata. The question is continually before them: To what extent does this real or apparent dip at the surface mean a similar dip

[1] Mather, Kirtley F., "Superficial Dip of Marine Limestone Strata, a Factor in Petroleum Geology," ECON. GEOL., Vol. XIII., No. 3, pp. 198–206, May, 1918.

in an oil sand far below the surface? The question arises also concerning the parallelism of shallow and deep sands and involves both direction and amount of dip. The unwary geologist is sure to be led into error, particularly where the general dips are low and, no matter how much attention to details is given, one is often puzzled. In reports, however, the reliable and usable field observations are likely to be discussed in some detail and most of the others ignored. The recording of anomalous dips and their classification should lead to some reduction in the percentage of errors though this percentage is already low.

As G. B. Richardson recently emphasized in a meeting of the petroleum club of Washington a small interval structure map based on complete information on a single bed over a considerable area, as for example that afforded by mine levels on a coal bed that has been mined out from an area a mile or more across has a different aspect from a carefully prepared map based on outcrops in the same area. In connection with this he discussed the very important question of what degree of detail is justifiable in the preparation of maps based on outcropping rocks, for the oil prospector.

There are not only the innumerable small peculiarities of each bed that tend to modify slightly the structure map but also occasional individual peculiarities of greater importance.

SOME STRIKING EXAMPLES OF ANOMALOUS DIPS.

Carnegie Swamp or Trench.—The Pittsburgh coal six to ten miles southwest of Pittsburgh lies in the form of a remarkably deep and narrow trench a map of which I incorporated in the Burgettstown-Carnegie folio of the United States Geological Survey. Detailed mine maps show that the swamp is several miles in length and ranges in depth from less than 20 to more than 100 feet. It is strikingly different from the broad synclines that are brought to light by a survey of outcrops, these having such gently dipping limbs and such broad flowing outlines as to be folds only in the technical sense. The trench seems to bear no relation to the structure of the surrounding country and it does

not seem possible that it was produced by plutonic or any other kind of diastrophic forces. Certain features of the coal indicate that the trench existed at the time the coal was laid down. The bed as a whole and each division or bench thickens toward the bottom of the swamp. This is true also of the "rooster" or "rider" coal lying several feet above the main bed. The somewhat branched outline of the trench and the fact that in some places one side of it and in other places the other side is the steeper, suggests an ancient valley, but the resemblance is not convincing. At one point the trench is crossed by a low anticline.

The upper or outer edge of the swamp is marked along part of its course by a hard sandstone roll and in certain steep portions of one side of the trench there are two or three such rolls. In the northwestern part of the O. I. C. mine the coal is faulted by a roll, the offset being about 10 feet. If this is an original depression the coal must have been deposited on a surface sloping as much as 10 degrees instead of the much more gentle slope of one degree or less that is generally assumed as the original attitude of most coal beds. In any case knowledge of the lay of the coal here is of little direct use in making a structure map of an oil sand far below the surface.

Valley Camp Swamp.—Along the railroad in the valley of Allegheny River near Valley Camp about 20 miles northeast of Pittsburgh are good exposures of a swamp or basin in the Upper Freeport coal. This swamp is much shallower than the one at Carnegie and does not differ materially from many other coal swamps in the region. The abundant exposures, however, offer unusual opportunities for measurements of thickness and dip of the coal and adjacent strata. The syncline here was original and although having a marked effect on thickness and lithologic character of the coal and associated strata it throws no light on the lay of the oil sands below.

Anomalous Dip Northeast of Valley Camp.—Still another type of anomalous dip is exemplified and well displayed among the excellent exposures on the east side of Allegheny Valley northeast of Pittsburgh. Two and a half miles northeast of Valley Camp

two layers of the Upper Freeport limestone have a dip of 10 to 15 degrees and terminate abruptly against an overlying horizontal layer. The lower strata may have been deposited on a steep surface on the side of a basin that was then filled and some of the filling material washed away before the upper bed of this limestone was laid down. No unconformity is recognized elsewhere at this horizon.

The McAlpin Arch in the Upper Bench of the Beckley Coal.—In the coal mine at McAlpin, West Virginia, the lower bench of the Beckley coal is nearly horizontal and of rather uniform thickness but the upper bench rises over a thick sandstone lens and descends on the opposite side to its usual position. Mine maps show a rise of 72 feet here in about a quarter of a mile. The especially noteworthy feature is the horizontal attitude of the lower bench.

Heaving of the Niagara Limestone at Chicago.—A portion of the Chicago drainage canal was cut through limestone and shortly after the quarrying operation was finished the solid limestone floor began to arch up in places. Randolph[2] says that he "found that the rock was moving up in seams right across the channel, sometimes nearly at right angles and sometimes diagonally. . . . At first we thought it was the expansion of the rock under exposure to the sun but it occurs in winter just as often as in summer, so that does not account for it."

Although this example of heaving in solid limestone of considerable thickness is indeed remarkable, it is only one of innumerable instances of the readiness with which stresses in sedimentary strata are eased because of the low degree of rigidity in the pile as a whole. Although we have immense caverns in limestone and lofty buildings constructed of this and other sedimentary rocks, the never ceasing tidal and barometric kneading of the earth and other factors evidently lead in the ages of geologic time to the closing of most large fissures and cavities and the adjustment by warping of other stresses particularly in rock with layers

[2] Randolph, Isham, "Drainage Canal," *Journal of the Western Society of Engineers*, Vol. II., No. 6, p. 690, December, 1897.

of clay shale. The result is of course additional irregularities of dip.

However, the formation of many if not of most sink holes does not involve local modification of dip. Mather refers to sink hole country in Kentucky and Tennessee where he says "few, if any, outcrops are in place." I have had occasion to examine many sink holes in Illinois, Missouri, Kentucky, and Tennessee, and in a large percentage of those in which the limestone is exposed it seems to be undisturbed. Apparently many if not most sink holes are not the result of collapsing of caves but of the underground removal of limestone residuum and other surficial material by way of a joint crack somewhat enlarged by solution. The underground stream channel becomes clogged from time to time causing ponds to form in the surface depression and occasionally the underground drainage is apparently rejuvenated and the sink hole drained by the continuous but gradual underground solution that takes place down to the varying depth at which the surface waters approach saturation with the calcium bicarbonate. Purdue came to a similar conclusion saying that the formation of sink holes by the falling in of roofs of caverns "is the rare exception."[3] He points out that the opposite view is prevalent among people in general and is explicitly stated by Gilbert and Brigham, Chamberlin and Salisbury, Hilgard and others.

Crossbedding on a Large Scale.—Another kind of dip that frequently puzzles the petroleum geologist comprises those that may be false dips of cross-bedded deposits. Where the exposures of such deposits are small, few, or poor, where the stratification is obscure or irregular, and where the crossbedding is on a large scale, the structure must often remain in doubt.

Can size and number of parallel beds be used in determining whether or not the apparent stratification is true? Attention is from time to time called to remarkably large scale crossbedding. Vaughan[4] describes examples of crossbedding simulating true

[3] Purdue, A. H., "On the Origin of Limestone Sink Holes," *Science*, new series, Vol. 26, p. 121, July 26, 1907.

[4] Vaughan, T. W., "A Contribution to the Geologic History of the Floridian Plateau," Carnegie Inst. Washington, Papers from the Tortugas Laboratory, Vol. IV., p. 176, 1910.

bedding in limestones of southern Florida. A few years ago D. E. Winchester[5] observed what he believes to be crossbedding on an unusually large scale in the Oligocene of northwestern Dakota. Here the false beds are about one foot thick, rise at an angle of 28 or 30 degrees a vertical distance of 20 to 100 feet and are practically parallel for a horizontal distance of 1,000 feet or more.

That a large part of the so-called chemical deposits is clastic as is shown by sorting, crossbedding, broken and even ground shells, etc., is becoming more generally recognized.[6] Such lime depositing waters as those around Florida and Yucatan are evidently transporting many more "ton-miles" of sediment than the Mississippi and the uneven bottom and the currents cause a great variety of anomalous dips. Only a minor fraction, consisting of oölite and some accumulations of shells, have not suffered much attrition or sorting.

Still, Mather's statement that "Obviously such a formation is to be avoided by the petroleum geologist" seems a little too strong. The greatest irregularity is near shore, and most shore deposits are re-worked before becoming buried.[7] Where the land is rising shore deposits and features are sure to be preserved for a time. A most striking example is to be seen a few miles north of Miami, Florida, where a long beach ridge of comminuted shells 10 to 20 feet high has been preserved and cemented into solid limestone. Although the cementation has been thorough, the clastic character of the original material and the bedding and crossbedding are still plainly evident in the walls of the quarries that have been opened in this impressive ridge.

As a rule calcareous deposits seem to become indurated more promptly than those of arenaceous or argillaceous make up,

[5] Winchester, D. E., "Crossbedding in the White River Formation of Northwestern South Dakota," *Jour. Geol.*, Vol. XXI., No. 6, pp. 550–556, Sept.–Oct., 1913.

[6] Shaw, E. W., "Significance of Sorting in Sedimentary Rocks," *Geol. Soc. Am. Bull.*, Vol. 28, No. 4, p. 925, Dec., 1917.

[7] Shaw, E. W., "The Rôle and Fate of Connate Water in Oil and Gas Sands" [discussion], *Am. Inst. Min. Eng.*, Bull. 103, p. 1451, July, 1915; *Trans.*, Vol. 51, p. 599, 1916.

though this depends to some extent on exposure to the air.[8] The writer has a pebble of limestone, the material of which was eight or ten years ago parts of the shells of living animals. The animals lived near Loggerhead Key, Tortugas; they died, their shells were broken up and incorporated in a beach that was being prograded at one part of the shore, the layers having an original seaward slope of about 10 degrees, they became indurated and were finally torn up by the waves and ground to pebbles—all in a few years.

Where the sea is advancing over the land and even where river sediments are accumulating, affording the conditions necessary for burial of deposits, there is always much reworking of the deposits, this process continuing until smothered by abundant sediment or deep water. For this reason shore deposits are ephemeral and most of the stratification that is preserved is, fortunately for petroleum geology, that of beds laid down at some distance landward or seaward from the shore, where fairly uniform conditions prevail over an area of considerable breadth.

GENESIS OF DIPS.

Classification.—It seems impossible to make a satisfactory genetic classification of anomalous dips by induction, until more have been studied and more descriptions have been recorded. On the other hand the subject may be approached deductively and from what is known concerning processes of deposition and deformation a genetic classification of all dips may perhaps be constructed.

GENETIC CLASSIFICATION OF CAUSES OF DIPS.[9]

I. Primary dips.
   a. Slope of bottom (results include crossbedding).
   b. Uneven thickness of beds.

[8] Vaughan, T. W., and Shaw, E. W., "Geologic Investigations of the Florida Coral Reef Tract," Year Book No. 14 of the Carnegie Inst. of Wash., p. 236, 1915.

[9] Perhaps for precision it would be better to speak of the dips of bedding planes than the dips of beds.

II. Secondary dips.
  a. Abyssal causes.
    1. Horizontal thrust.
    2. Uneven uplift or subsidence.
  b. Hypabyssal causes.[10]
    1. Irregular settling including the results of irregular compacting and of the solution and precipitation involved in irregular cementation.
    2. Flowage of clay and differential movement of jointed and fractured rock due to various differential stresses including those of a hydrostatic nature.
  c. Surficial causes.
    1. Irregular underground solution including both the solution of limestone along underground streams and the irregular solution of other rocks by ground water.
    2. Creep induced by gravity and by temperature and chemical changes, plant and frost action, etc.

Apparently all the secondary causes may, with or without the coöperation of kneading,[11] be effective in producing differential movement. Apparently also the tidal and barometric stresses to which the earth's crust is continually subjected are sufficiently powerful to cause some kneading and afford opportunity for all stresses to become eased.

REMARKS CONCERNING THE CAUSES OF DIPS.

Slope of Bottom.—It is necessary first to make sure that the apparent stratification is original, whether true or false, and not due to secondary causes such as some character developed at

[10] Hypabyssal: introduced by Brögger about thirty years ago; was and has since been applied largely to *rocks* found between the zone where coarse-grained and holocrystalline rocks develop, and the surface where effusive lavas congeal, but it is hoped that petrologists will allow its use for a position as well as for a class of igneous rocks, even though the exact position has never been defined, and will also permit its use in the great sedimentary basins.

[11] Shaw, E. W., "Crevices and Cavities in Oil Sands" [discussion], ECON. GEOL., Vol. XIII., No. 3, pp. 216, 217, May, 1918.

various positions occupied at different times by the top of ground water. The original slope of the bottom is concerned not only with the general undulatory form of the bottoms of all bodies of water but with crossbedding—the results of contemporaneous or alternating cut and fill, with beds laid on the flanks of submerged hills, and with contemporaneous deformation. Most of the slopes are temporary and cause anomalous dips.

Uneven Thickness of Beds.—Similarly the uneven thickness of beds applies not only to the general lawless irregularities, but to the regular thinning of beds and formations in a certain direction; to the top set, fore set, and bottom set of beds of deltas; to the infrequently preserved strata of beaches and dunes; and to the more or less lenticular beds of sand, shell fragments, and algal secretions that are piled one on another until islands are formed. These also are not very often preserved and many of those in waters inhabited by corals are falsely called coral reefs or islands.

The fact that few beds approximate uniformity in thickness and that the thick portions of beds are irregularly distributed vertically and horizontally give rise to many anomalous dips though the effects are apparently much subdued by the relatively even subsidence of the general sedimentary basin, for the piling up of beds of irregular thickness at random is in a way a self-checking process if the basin subsides evenly and is kept nearly filled with deposits.

Horizontal Thrust.—Horizontal thrust has evidently been much more powerful in some oil regions than in others and the effects on dip depend on the amount of pressure, on the competence and attitude of the beds, and on differences in the resistance of various beds. Differential movement involves flowage of soft beds and jointing, fracturing, or brecciation of hard beds. The stresses and effects have recently been discussed by Chamberlin and Miller.[12]

Gardner[13] and McCoy[14] urge that the low anticlines far out in

[12] Chamberlin, R. T., and Miller, W. Z., "Low-angle Faulting," *Jour. Geol.*, Vol. XXVI., No. 1, pp. 1–44, Jan.–Feb., 1918.

[13] Gardner, J. H., "The Vertical Component in Local Folding," *S. W. Assoc. Petrol. Geol.*, Bull. 1, pp. 107–109, 1917.

[14] McCoy, A. W., "The Vertical Component in Local Folding" [discussion], *S. W. Assoc. Petrol. Geol.*, Bull. 1, pp. 109–110, 1917.

sedimentary basins are due to vertical uplift and that the rocks are not competent to communicate vertical thrust to any great distance. The present writer is inclined to agree with them though for reasons that differ somewhat from those they assign. The habit of regarding low anticlines as due to lateral pressure is unfortunate,[15] but still it seems probable that when the pre-Cambrian rocks move laterally, the strata above are carried along to a certain extent, being affected by pressures whose directions lie in all parts of the quadrant from horizontal to vertical and that the kneading process leads to the equalization of lateral pressure through warping.

Uneven Subsidence and Uplift.—Uneven subsidence at various intervals may cause considerable departure from parallelism as well as modification of the dip of all beds but the writer has been impressed with the idea that except for broad variations the subsidence is generally even. As a result we find the somewhat surprising phenomenon that a single stratum or a small group of strata is several hundred per cent. thicker in some places than in others, but that commonly a large group of strata is, where thickest, only a few per cent. thicker than where thinnest. For example, in Washington County, Pa., the top of the Pittsburgh "rooster" coal is in some places only two feet above the top of the main coal and elsewhere; only a mile or so away the distance is 24 feet or 1,200 per cent. greater. On the other hand the spacing of the Pittsburgh coal and the hundred foot sand nearly 2,000 feet below does not vary by more than about 1 per cent. per mile and is commonly much less. The demands of the doctrine of chances alone would call for much greater convergence.

At the same time the spacing of beds far apart does certainly vary somewhat on account of irregular subsidence of the basin floor or irregular uplift at such times of positive movement as there may have been in each of the great geosynclines.

Irregular Settling.—In many regions far from mountains irregular settling seems to have been the dominant factor in the

[15] Shaw, E. W., "The Rôle and Fate of Connate Water in Oil and Gas Sands" [discussion], *Am. Inst. Min. Eng.*, Bull. 103, p. 1454, July, 1915; *Trans.*, Vol. 51, p. 606, 1916.

development of such structural features as are found. An excellent example is believed to be furnished in parts of northwestern Pennsylvania where the strata are nearly horizontal and structure contours wander about without much system. "These structural features are the product (1) of irregularities of the surface upon which each layer was deposited; (2) differential settling; (3) warping at various times since deposit."[16]

An anticline may conceivably be formed by irregular settling if near the base of the sedimentary pile there is a thick sand of no great breadth, for sand on the average probably loses not more than 25 per cent., whereas clay may lose as much as nine tenths of its original volume and coal beds shrink still more. According to Norton[17] salt domes would fall in this category, the salt being non-yielding and the surrounding materials settling down considerably. Well-formed anticlines and domes due to irregular settling are, however, probably not numerous, and irregularities of solution and cementation are probably a still less active factor, though both undoubtedly serve to modify the structure arising from other causes.

Flowage.—That various kinds of stresses are often eased by flowage of clay seems certain notwithstanding the fact that some shales, particularly hard or sandy shales, have almost undisturbed lamination and many beds of clay shale have fairly uniform thickness. Clay veins in coal and occasional other indications of internal motion show that flowage is a factor to be considered in the study of the causes of the relatively slight departures from horizontality, displayed by the strata in many oil fields. Consolidated rocks may also suffer some differential movement along joint, fracture, and brecciation surfaces.

The possible causes of the stresses are numerous. Hydrostatics is probably in places a factor. Migration of isogeotherms, due particularly to subsidence, sedimentation, uplift and erosion,

[16] Shaw, E. W., and Munn, M. J., "Coal, Oil, and Gas of the Foxburg Quadrangle, Pa.," U. S. Geol. Survey Bull. 454, p. 40, 1911.

[17] Norton, E. G., "The Origin of the Louisiana and Eastern Texas Salines," *Am. Inst. Min. Eng., Trans.*, Vol. 51, pp. 508–511, 1916.

plays a part. McCoy[18] believes that capillary pressures become very great—possibly so great as to cause anticlines but Washburne[19] shows clearly that he is mistaken and that the stresses due to the capillary pressures developed by the surfaces between oil, gas and water in the earth are of a relatively low magnitude.

Irregular Underground Solution.—Although as noted above the collapsing of cavern roofs does not seem to be a widespread cause of anomalous dips, solution evidently plays a very large rôle in modifying the dips of rocks at the surface. It could not be otherwise in view of the facts (1) that the strata in any single locality usually show a considerable range in solubility and (2) that underground solution is everywhere an active process, for all ground and stream water have much mineral matter in solution. Although quartz sand is generally regarded as fairly insoluble, under most natural conditions the waters draining areas of quartz sand, even those in which alkalies are scarce, carry a considerable burden of silica. The writer is inclined to surmise that many undrained depressions in sandy country have been developed in spots where the solution of quartz sand has proceeded with unusual rapidity, the process being perhaps self-accelerating, the water from the higher ground flowing to the depressions and there remaining to take on a maximum load in solution. The sand and other soil matter in the depressions is wet a large part of the time and more water comes in contact with the particles than with those of the upland soil. Examples are to be seen in southern Alabama, and in central New Jersey.

Creep.—Besides the more generally recognized features of hillside creep, are certain effects that have not received much attention. In many places hillside surficial material seems to be creeping up instead of down—perhaps due to swelling by weathering of the surface portions of certain underlying beds. Excellent and numerous examples are seen where coal-bearing formations

[18] McCoy, A. W., "Some Effects of Capillarity on Oil Accumulation," *Jour. Geol.*, Vol. XXIV., pp. 798–805, 1916.

[19] Washburne, C. W., "Discussion of Some Effects of Capillarity on Oil Accumulation," by A. W. McCoy, *Jour. Geol.*, Vol. XXV., No. 6, pp. 584–586, Sept.–Oct., 1917.

lie nearly horizontal in hilly country. Openings in the coal beds distributed along their outcrops commonly show that the beds dip down into the hill for 10 or 20 feet. Elsewhere the coals are wavy at the outcrops as though affected by two opposing forces, one tending to pull the outcropping edges down, and the other to push them up.

In many conical hills of shale, especially those capped by sandstone or limestone, the shale seems to be thinner than where it lies far below the surface, suggesting that in the outlier type of occurrence the shale has flowed laterally, letting the cap rock down.

The recording of examples of various kinds of anomalous dips and the study of the possible causes of dips should be of assistance in the elimination of errors in structure maps.

RACEWINITE: A PECULIAR MINERAL FROM ORE DEPOSITS IN UTAH.

A. N. WINCHELL.

In the latter part of the year 1917 Mr. H. V. Winchell sent to the writer from Utah a sample of a mineral which has some unusual characteristics. It was obtained on the thirteenth level of the Highland Boy mine at Bingham, Utah, about fifteen feet south of a large mass of porphyry in the Highland Boy limestone. Underground in the mine it was regarded as probably a silicate resembling chlorite.

It has since been found in many places on several different levels of the Highland Boy mine and is a common mineral in the district. It is apparently one of the latest products of hydrothermal alteration of the limestone, being in veins and irregular masses in the country rock, and also replacing the carbonate portion of calcareous sandstones. It is often intergrown with pyrite and other ore minerals, as well as with a brown isotropic mineral which resembles garnet, but is much softer. It reabsorbs water so vigorously that attempts to prepare thin sections have been frustrated by the expansion of the mineral upon taking up water (after heating in Canada balsam) which broke the fragment loose from the glass.

When the first sample was received it was found to have a brown to yellow color in thin splinters, and a nearly black color in mass; the luster varied from dull to bright, like brownish black coal. But a freshly broken piece disclosed an interior color which was bluish green, changing slowly in air or in water, in sunlight or in the dark, to the brownish black color. When first received the color change had penetrated unfractured pieces to a depth of about one millimeter; in six months it penetrated to the center of the largest pieces. In boiling nitric acid the black mineral is insoluble but changes color gradually to a translucent

brownish yellow; it is unchanged and unattacked in sulphuric acid; it is slowly soluble in hydrochloric acid, the unattacked portion changing color as in nitric acid but more slowly.

The very fine powder of the mineral is nearly white with a yellowish tinge; particles under the microscope are pale to brownish yellow; upon immersion in clove oil the color changes to green gradually, becoming black except on the thinnest edges which remain grass green. Immersion in cedar oil, anise oil, or castor oil, produces no change of color, but in a solution of one part of cedar oil in seven parts of clove oil the pale yellow particles change first to grass green and then to reddish brown. In a solution of one part of cedar oil in two parts of clove oil the change to deep green occurs as before, but the second change is not completed.

These color changes were wholly unexpected and various attempts were made to discover a cause. Tests for an organic constituent gave only negative results; unlike sepiolite the mineral does not give a burnt odor upon ignition.

The mineral has a hardness of about 2.5; it is rather fragile, some pieces having been broken in the fingers. The specific gravity is 1.94 to 1.98 as determined in petroleum on various pieces. Determination of the specific gravity in water yields the same result, but is accompanied by a slacking or breaking up into many small pieces. After six months at Madison the breaking of large pieces upon immersion in water occurs with a sharp audible sound. The mineral adheres to the tongue, but slacks if left upon the tongue.

Under the microscope the mean index of refraction has been determined as about 1.51 by immersion in liquids of known index. Both indices are higher than 1.50 and lower than 1.53; they change appreciably as the color changes. The birefringence is comparable with that of quartz. The mineral is negative and biaxial with a large optic angle. It is coarsely crystalline with no visible cleavages or crystal faces, but it has well marked conchoidal fracture.

Qualitative chemical tests made by H. Gordon Taylor indicate

the absence of chromium, zinc, manganese, nickel, cobalt, barium, strontium, and phosphoric acid, and the presence of silica, alumina, iron oxide, lime, magnesia, and water. Spectroscopic tests made by Professor Louis Kahlenberg indicate the absence of rare earths.

Quantitative analyses have been made by Philip J. Hickey at Madison and Charles L. Austin at Bingham. Their results are respectively as follows:

SiO_2	43.92	43.24
Al_2O_3	23.68	23.69
Fe_2O_3	7.37	8.05
MgO	.50	.78
CaO	2.52	2.42
H_2O	22.04	21.80
	100.03	99.98

A separate determination by Mr. Austin indicates that a small part of the iron may be in the ferrous state; he obtained as much as 0.60 per cent. of FeO from one sample of the green mineral fresh from the mine.

The mineral contains a large amount of water much (or all?) of which can be driven off, by heating to a dull red temperature, without destroying the crystal structure or materially altering the optical characters. Tests made by Mr. Hickey show that the less of water is 8 per cent. at 70 degrees centigrade, 10.7 per cent. at 110, 1.8 per cent. at 165, and 12.1 per cent. at 200 degrees. The water is lost by exposure merely to dry air, as shown by the following test made by Mr. Austin. Two samples placed

Date.	Large Sample Weight in Grams.	Small Sample Weight in Grams.
August 21	5.2350	1.5424
" 22	4.8757	1.4116
" 24	4.7690	1.3857
" 26	4.7130	1.3580
" 27	4.6965	1.3570
" 29	4.6617	1.3498
" 31	4.6458	1.3461
September 14	4.5885	1.3320
November 9	4.5618	1.3243

in a dessicator (dried by H_2SO_4) lost 7 to 8 per cent. of water in one day; the larger sample lost its water more slowly than the smaller one, but both continued to lose water at a gradually decreasing rate as long as the experiment continued. The results may be tabulated as follows:

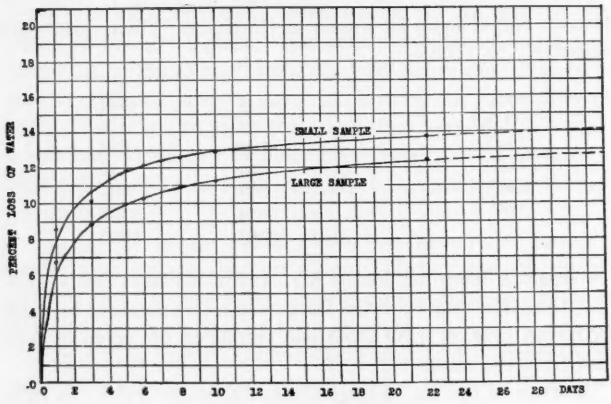


FIG. 58. Diagram showing the rate of loss of water of two samples of racewinite dried in a dessication at ordinary temperature.

The results (except the last weighings) are shown graphically in the figure, from which it seems probable that all the water would be lost under these conditions if sufficient time were allowed, though the time necessary would doubtless be measured in months, if not in years. After 49 days the small sample had lost 14.14 per cent. of water and the large sample 12.86 per cent. During weighing of the samples their weight increases rather rapidly on account of reabsorption of water, and it is probable that all the water may be taken up again. Therefore it is reasonable to believe that the water is present in solution in the silicate, as in zeolites. After fifty days in a desiccator the mineral seems to take up water more slowly on exposure to moist air and to be somewhat lighter in color.

Various theories have been considered to explain the remarkable color changes exhibited by this material, but so far without satisfactory results. In the absence of rare earths and cobalt, attention has been concentrated on the iron. The first change from bluish green to black may perhaps be due to oxidation of the small amount of ferrous iron in the mineral; this change is retarded by keeping the mineral in mine water (which contains some sulphuric acid). The change to brownish yellow upon boiling in dilute nitric acid is apparently due to extraction of considerable iron from the mineral without destroying the crystalline character nor radically modifying the physical characters. This supposition is being tested at present. It implies that the iron, as well as the water, is present in solution in the silicate, and such a condition is so extraordinary that the idea will be tested as thoroughly as possible.

Study of the mineral as it occurs underground in the mine suggests that it varies by insensible gradations into a lighter colored variety which probably contains less iron. In thin sections it seems to exhibit a corresponding variation in refringence and birefringence, both increasing with increase of iron. But these variations need further study.

Since the physical characters and chemical composition of this mineral show that it is unlike any previously described, it is a new species, for which the writer would suggest the name *racewinite,* from the cable address (Racewin) of H. V. Winchell, who recognized the substance as a peculiar mineral and collected material for its study.

DISCUSSION

This department has been established by the editors in order to afford to those interested in questions relating to economic geology an opportunity for informal discussion. Contributions are cordially invited either in the form of discussion of more formal papers appearing in earlier numbers or bearing upon matters not previously treated. Letters should be directed to the Editor, Sheffield Scientific School of Yale University, New Haven, Conn. The full name of the author should be attached to all communications.

ORIGIN OF THE CAP ROCK OF THE GULF COAST SALT DOMES.

Sir:—I believe that Mr. Rogers's paper on "The Intrusive Origin of the Gulf Coast Salt Domes," published in the September number of this journal for the current year, marks a commendable advance in our study of the salt dome problem. This is true not only for his strong presentation of the tectonic theory but also for his emphasis of the probability that the origin of the cap rock has been distinct from and subsequent to that of the salt stock of the domes.

The form of the explored part of the salt masses of the domes is generally that of the uppermost part of a truncated cone with steeply pitching sides. In many of the domes, the top of the salt mass is gently rounded. The cap rock occurs generally as a tip or wafer covering the top of the salt stock or extending thimble-like over the top and down the sides of the salt. The Sour Lake, Texas, dome and several of the Mexican domes are notable examples where the cap rock extends over the top and far down the flanks of the salt.

This thimble-like form of the cap rock seems to indicate that it was formed upon and took its shape from the salt mass already in place. This condition and the fact that the cap rock is altogether lacking from certain domes, notably those explored to date

in east central Texas, favor the separate origin suggested by Rogers.

Although this hypothesis of separate origin was set forth in an exposition of the tectonic theory of salt dome origin, it seems to apply equally well to any of the previous theories which postulate the formation of salt masses by the deposition of salt from solutions.

The native sulphur, natural gas and petroleum which are commonly found in greater or less amounts in the cap rock of the domes of the Gulf Coast could only have attained their present position after the formation of the cap rock.

On these bases, the sequence of geologic events seems to have been: (1) the formation of the salt masses; (2) the formation of the cap rock; and (3) the accumulation of petroleum, natural gas and sulphur in the cap rock.

Accepting such order of events in salt dome formation, the tenable theories to account for the cap rock material would seem to be limited to: (1) residue left by the solution of the upper part of the salt stock; (2) casual encounter of gypsum or limestone in the sedimentary rocks by the intruding salt, as suggested by Rogers; and (3) deposition from solution.

In view of the small percentage of minerals of cap rock type which are present in the salt masses, it seems that several hundreds or thousands of feet of the salt stock would have had to have been dissolved in order to leave a residue quantitatively sufficient to account for the great thicknesses of cap rock present in many of the domes.

Any such wholesale removal of salt would have resulted in a letting down of the overlying strata and perhaps in a caving-in of the wall rock. There is no evidence that such considerable movement has taken place. In many of the Mexican domes, the strata over the salt and cap rock contain somewhat lenticular but well marked and more or less persistent petroleum-bearing sands which can be traced from one well to the next. These sands show no evidence of downward movement. In fact, except for the small lakes on several of the Gulf Coast domes, there is no evidence of subsidence. These lakes are doubtless often due to

sinking of the covering strata to fill space made by the dissolving of the salt but such small movements as they indicate are not adequate as evidence of solution of the salt on the scale necessary to the residue theory of cap rock formation.

The encountering of beds of gypsum or limestone in the sedimentary rocks by the intruding salt mass might satisfactorily explain a few isolated occurrences. Mr. E. G. Woodruff has told me that he strongly suspects the cap rock at Sour Lake to be of sedimentary origin. In such event, it could only have come to its present position after the manner suggested by Mr. Rogers.

This theory, however, would not seem to be generally applicable. Nature may be casual but to depend upon such casual encounter as a sufficient explanation for a phenomenon of such common occurrence as that of the cap rock over the salt, would seem to be in direct disregard of the laws of probability and chance.

There remains for consideration only the possibilities of deposition of the cap rock by precipitation from solution. In consideration of any such theory, we are concerned with the source of solution, both solvent and material dissolved, the agency causing precipitation, and the quantitative sufficiency of the suggested precipitation.

I shall not consider the various possible combinations which might suggest themselves but shall outline briefly a theory which seems to be more acceptable than those yet proposed.

That the degree of solubility of various compounds in water is considerably modified by the presence of certain other compounds is well known. Frank K. Cameron,[1] in his studies of the chemistry of alkali soils, has investigated the solubilities of calcium sulphate and calcium carbonate in aqueous solutions of certain electrolytes, notably sodium chloride, at various temperatures. He found that the solubility of gypsum, which is approximately 2.41 grams per litre in pure water at 0° C., increasing to 2.72 grams at 38° C., then decreasing to only 2.22 grams at 99° C.,

[1] Various papers in bulletins 18, 33 and 49, U. S. Dept. Agriculture, Soils Division and in Volume 5, *Journal of Physical Chemistry*, 1901. These papers give numerous references to the bibliography of solutions.

increases greatly in aqueous solutions of sodium chloride at various temperatures.

Experimenting with natural gypsum, which had been carefully analyzed for purity, he got the interesting results expressed in the following table.

SOLUBILITY OF CALCIUM SULPHATE IN AQUEOUS SOLUTIONS OF SODIUM CHLORIDE AT 23° C.

NaCl.	$CaSO_4$.	Gypsum.	NaCl.	$CaSO_4$.	Gypsum.
.99	2.37	2.99	129.50	7.50	9.42
4.95	3.02	3.82	197.20	7.25	9.17
10.40	3.54	4.48	229.70	7.03	8.88
30.19	4.97	6.31	306.40	5.68	7.19
49.17	5.94	7.51	315.55[3]	5.37[3]	6.97[3]
75.58	6.74	8.53			

[2] Cameron, Frank K., "Solubility of Gypsum in Aqueous Solutions of Sodium Chloride," U. S. Dept. Agriculture, Division of Soils, Bull. No. 18, pp. 25–45, 1901 (Table IX.).

[3] The solution in this case was in contact with both gypsum and sodium chloride in the solid phase.

This table shows a maximum solubility for gypsum of 9.42 grams per litre in a solution containing 129.5 grams of sodium chloride per litre and a decrease in solubility of 2.45 grams of gypsum with an increase of sodium chloride in the solution to near the saturation point. Mr. Cameron concludes that the gypsum has a maximum solubility in solutions containing 134–140 grams of sodium per litre.

Similar experiments at higher temperatures were repeated with a narrower range of sodium chloride content but the points of maximum solubility for gypsum were not determined.

Experiments in solubility of calcium carbonate in aqueous solutions of sodium chloride in equilibrium with atmospheric air were also made and the calcium bicarbonate formed in the solution was found to have a maximum solubility of .101 gram per litre in a solution containing 39.62 grams of sodium chloride per litre as compared with solubilities of .06 and .04 gram per litre in

solutions containing respectively no sodium chloride and 267.60 grams of sodium chloride per litre.[4]

These discoveries of the variation in solubility of calcium sulphate and calcium carbonate in aqueous solutions of salt suggest most strongly that the cap rock of the salt dome, consisting chiefly of gypsum, anhydrite and limestone, was formed by precipitation resulting from an increase in salt content of the solution upon its coming in contact with and dissolving part of the salt stock of the dome. The solution from which the cap rock material was precipitated was apparently that of circulating waters which had taken into solution sodium chloride and calcium salts encountered in its passage through the sedimentary rocks.

The sedimentary deposits of the Coastal Plain contain ample gypsum, limestone, salt and circulating water to account for the formation of the cap rocks, and since the action suggested is one which could go on continuously with the circulation of surface waters, perhaps being operative at the present time, there is no reason to discard the theory because of quantitative insufficiency.

If one assumes that the cap rocks, as originally deposited, were largely calcium sulphate, the theory of derivation of the sulphur of the domes through reduction by hydrocarbons has an added attractiveness.

In presenting this theory I have dealt only with simple solutions of such materials as quantitatively far outrank the other dome minerals. The actual solutions and geochemical transformations at various temperatures and pressures are probably far more complex.

E. DEGOLYER.

[4] See Cameron, Frank K., and Seidell, Atherton, "Solubility of Calcium Carbonate in Aqueous Solutions of Certain Electrolytes in Equilibrium with Atmospheric Air," U. S. Dept. of Agriculture, Division of Soils, Bull. No. 18, pp. 58-64, 1901. Also *Journal of Physical Chemistry*, Vol. 5, p. 643, 1901.

INTRUSIVE ORIGIN OF THE GULF COAST SALT DOMES.

ITS BEARING ON THE ACCUMULATION OF OIL.

Sir:—G. Sherburne Rogers's interesting paper on the salt domes of the Texas-Louisiana Gulf plain in the September issue of ECONOMIC GEOLOGY, in which the origin of these domes is attributed to the forcing up of masses of plastic salt from a hypothetical bed of salt in deeply buried strata, throws considerable light on the occurrence of oil on some of these domes and Rogers's arguments seem so conclusive that it may not be out of place to critically examine the generally accepted ideas in regard to the oil accumulations.

The adherents of the structural ("anticlinal") theory of oil accumulation have always pointed to these domes as examples of true dome or quaquervasal accumulation. Now it is evident that the disturbance of the fairly soft strata into which the salt masses have been intruded is very limited in extent and merely amounts to a turning up of the broken edges of the punctured strata immediately adjoining the salt masses. The strata are not resistant enough to be arched over a wide area. Thus it seems that the area from which the oil would migrate due to gradient around the domes is altogether too limited to quantitatively account for the oil.

Just as Rogers's arguments require a bed of salt deeply buried in late Paleozoic or early Mesozoic strata may we not also suppose that beneath the surface are oil-bearing formations such as the Cretaceous of the Caddo field of Louisiana and other districts in Texas. Rupture of these formations by the intrusion of the salt masses would form open channels, and the pushing of the salt mass through the upper softer beds would permit the free circulation of liquids through the jagged and broken zones around the salt masses. The oil from the lower accumulations would thus move more or less vertically in open channels and would accumulate finally in such porous material as is found at the tops of the so-called "domes."

The fact that many of the domes are barren of oil would simply mean that oil only occurs where the intrusion of the salt happened to penetrate a previously existing accumulation of oil. The other requirement of course is the existence of some form of impervious cap above the salt mass to prevent the escape of the oil. By this conception, doming of the strata is not a necessary requisite for accumulation. If the oil migrates and accumulates due to such arching of the strata as may exist, it is not only difficult to quantitatively account for the oil due to the limited area of disturbance, but it is also difficult to see why more of the domes do not contain oil.

WILLIAM F. JONES.

OCCURRENCE OF THE SILVER HALIDES IN THE OXIDIZED ZONE OF ORE DEPOSITS.

Sir:—In regard to the occurence of the silver halides in ore deposits it has appeared to me that the seemingly anomalous vertical order in which they occur, namely that silver iodide, which is 1,400 times as insoluble as silver chloride, should occur below the zone in which the chloride predominates, is most probably the result of the reversible reaction between ferrous and ferric iron and iodine and iodides. As this possibility has not been previously suggested, it is perhaps worth while to set forth the chemistry of the process.

It has been established by Seubert and Rohrer[1] that an equilibrium subsists between ferous and ferric iron and iodine and iodides. They studied the following reaction (and others of similar type) and found it to be reversible:

$$Fe_2(SO_4)_3 + 2KI = 2FeSO_4 + K_2SO_4 + I_2.$$

The reaction written ionically is:

$$2Fe^{++} + 2I \rightleftharpoons 2Fe^{++} + I_2,$$

[1] Seubert, K., and Rohrer, R., "Ueber die Einwirkung von Ferrisulfat auf Jodkalium und Jodwasserstoff," *Zeitschr. anorgan. Chem.*, vol. 7, p. 137, 1894.

where I is ionic iodine and I_2 is molecular iodine. For the condition of equilibrium we have:

$$\frac{(Fe^{+++})^2 \times (I)^2}{(Fe^{++})^2 \times (I_2)} = K.$$

The constant K has been computed by Stieglitz[2] to be 5.6×10^{-5}. "It is evident from the value of the constant," says Stieglitz, "that the chief tendency of the reversible reaction will be toward the reduction of the ferric ions and the liberation of iodine."

Meteoric waters descending through the outcrop of an oxidizing ore body will carry an amount of iron in solution enormously in excess of the iodine present, and in the oxidized zone ferric iron will predominate. Therefore, the concentration of ionic iodine in these solutions will be so minute as to be insufficient to precipitate the silver as AgI. Furthermore, if the solutions contain free sulphuric acid, the acid would react with any iodides present, liberating hydroiodic acid. Now hydroiodic acid is extremely sensitive to oxidation by atmospheric oxygen, yielding water and free iodine.[3] This reaction leads thus to the same result as that brought about by the presence of ferric salts.

From the standpoint of the geologist both reactions (that due to ferric salts and that due to free acid) are expressions of a single geologic fact, namely, that the solutions circulating in the outcrop of the vein are in an oxidizing environment and that in this environment the iodine ion becomes suppressed. This suppression of the iodine ion may proceed so far, depending on the relative concentrations of ferric iron and iodine, that the iodide of silver will not be precipitated in the zone of active oxidation. In this zone part of the silver, however, will be precipitated as chloride.

As the solutions migrate downward, however, the ferric salts are precipitated as basic compounds (jarosite) or as hydroxides as a result of hydrolysis due to decreasing acidity of the solutions, or they become reduced to the ferrous state by reaction with the

[2] "Qualitative Chemical Analysis," Vol. I, Theoretical Part, p. 274, 1912.
[3] Stieglitz, Julius, *op. cit.*, pp. 306–307.

primary sulphides of the ore body. The reaction

$$2Fe^{+++}2I \rightleftharpoons 2Fe^{++} + I_2$$

can then proceed to the left; and as soon as the concentration of iodine ions thus formed exceeds the amount determined by the solubility-product of AgI, precipitation of silver iodide will ensue. Iodine ions will thus be removed from the solution, and the reaction can proceed to the left practically to completion. The whole reaction may therefore be written:

$$2Fe^{++} + 2Ag^{+} + I_2 = 2AgI + 2Fe^{+++}.$$

According to this analysis of the problem, then the hypsometric order of the silver halides is determined by the fact that ferric compounds predominate in the upper zones of oxidizing ore bodies but become gradually reduced to ferrous compounds as the solutions move downward. From these considerations it appears that AgI is most likely to occur in the zone of secondary sulphides.

ADOLPH KNOPF.

SCIENTIFIC NOTES AND NEWS.[1]

WILLET G. MILLER has recently received appointment as member of the Committee of Gold Producers of the British Empire, at London.

J. F. KEMP is in Oregon.

J. B. TYRRELL has been visiting the Bourkes and Miller-Independence mines and gold prospects in northern Ontario.

EDMUND OTIS HOVEY, secretary of the Geological Society of America, was recently a visitor in New Haven.

S. N. GRAHAM and G. C. MACKAY recently joined the professorial staff of the Faculty of Science, Queens University.

AUGUSTUS LOCKE is at present in the East for a month or so.

WALTER E. GABY, who, for the past year has been connected with the Nevada-Douglas Cons. Co., has opened an office in Salt Lake City. Mr. Gaby will engage in engineering geological work.

OLAF P. JENKINS has left the State College of Washington, and is now geologist of the Arizona Bureau of Mines, University of Arizona, Tucson, Arizona.

GEORGE H. GARRY is at Tonopah, Nevada.

ANDREW C. LAWSON, professor of geology in the University of California, has gone to France, to serve with the Red Cross.

FRANK L. PROBERT, professor of mining in the University of California, has gone to Washington, having been appointed a member of the National Committee on Minerals.

PHILIP ARGALL has been in Rico, Colorado.

[1] Geologists, mining engineers and others interested in applied geology are invited to keep the editor informed of new investigations of mining districts or scientific studies undertaken by them, together with such other scientific and personal items as may come to their notice.

T. SKEWS SAUNDERS, manager of the Dos Estrellas, has returned from London and Paris to El Oro, Mexico.

STANLEY A. EASTON and LOUIS S. CATES have been nominated directors of the American Institute of Mining Engineers.

F. J. SUR, petroleum geologist, has returned to his Denver office after five months spent in inspecting properties for many different companies throughout Montana and Wyoming.

E. L. BRUCE, of the Canadian Geological Survey, is examining the Star Lake District, Manitoba. Discoveries of molydenum and scheelite have been made at this place.

R. E. SOMERS is leaving Cornell University to become a member of the firm of Johnson and Huntley and also to give some lectures in the University of Pittsburgh.

JOHN SEWARD, with rank of major, has been assigned to the executive division of the General Staff at Washington. He will study the subject of war minerals.

H. W. ALDRICH has become assistant general superintendent of the Inspiration Consolidated Copper Co.'s plant in Miami, Ariz., having resigned from his position with the Anaconda Copper Mining Co.

C. E. RHODES has recently been presented with the medal of the Institution of Mining Engineers.

W. F. WHITE has been visiting the English Provinces, stopping at Birmingham, Sheffield, Stockton, Middleborough and Leeds.

A. G. HAULTAIN, with rank of captain, has joined the Engineering Unit of the Canadian Expeditionary Force to be sent to Siberia.

FRED HELLMAN and F. S. SHAW are inspecting mines in Mexico, controlled by Messrs. Guggenheim.

J. AUSTEN BANCROFT, of McGill University, who has been at Anyox, B. C., in connection with geological work has returned to Montreal.

B. L. CUNNINGHAM has accepted the position of consulting geologist for the Doheny Pacific Petroleum Co., with headquarters in Los Angeles. Mr. Cunningham was formerly assistant geologist to the Southern Pacific Co.

E. T. CORKILL succeeds Mr. J. C. Nichols as superintendent of mines for the International Nickel Co., at Copper Cliff, Ontario.

STEPHEN HARRIS has accepted the position of manager of the Mount Cuthbert Copper Mines, North Queensland.

H. R. VAN WAGENEN has received the appointment of general manager for the Canada Copper Corporation, at Copper Mountain, British Columbia.

CHARLES R. VAN HISE, president of the University of Wisconsin, died in Milwaukee, November 19th. Professor Van Hise had recently returned from Europe where he was engaged in the study of problems of reconstruction.

THE government restrictions on the use of platinum, iridium and palladium were modified on November 14 by the director of the Bureau of Mines so that licenses are not longer required to handle them, and their use for non-essential purposes is increased.

CONGRESS has authorized the President to control until two years after the end of the war, the supply and distribution of the following: Antimony, arsenic, ball clay, bismuth, bromine, cerium, chalk, chromium, cobalt, corundum, emery, fluorspar, ferrosilicon, fullers' earth, graphite, grinding pebbles, iridium, kaolin, magnesite, manganese, mercury, mica, molybdenum, osmium, sodium, platinum, palladium, paper clay, phosphorus, potassium, pyrites, radium, sulphur, thorium, tin, titanium, tungsten, uranium, vanadium and zirconium.

INDEX TO VOLUME XIII.

[NOTE.—In this index the titles of principal papers and the headings of departments, as *Discussion*, are in italics.]

Abrams, D. A., on compressive strength of concrete, 593; on wear tests on concrete, 564
Abrams, D. A., and Harder, O. E., on testing organic impurities in sand, 589
Abrasion tests of gravel, 588; of sand, 593; of stone, 565, 567
Absorption of water by stone, 575
Adams, F. D., on deformation of rocks under pressure, 473
Adhesion, 340, 350
Adsorption, 339, 550
Akita oil district, Japan, 516
Allen Co., Ky., geology, 204
Allocation of war minerals, 531
Alteration, in Telkwa district, B. C., 354, 366; of granite, 6
Aluminum, 502
Alvarado mining claims, 248, 257, 272
Ames, J. S., on geology in the Great War, 145
Ammonia sulphate, 510
Analyses (*see* Chemical analyses)
Andalusite crystal, 256
Andalusite hornfels, 248
Andrews, E. B., on the geological relations of rock oil, 210
An Ka Roo Kai granite boss, 7
Anomalous dips (Shaw), 598
Anomocline, 598
Anticlinal theory of oil and gas accumulation, 208, 318; date of origination, 219
Antimony, 501
Aranzazu, gold occurrence, 259
Archean rocks, Korea, 9
Arendal, Norway, ilmenite from, 423
Arnold, Ralph, and Anderson, Robert, on the accumulation of oil, 211
Arrhenius, S., on salt deposits, 480
Arsenic, 501
Asbestos, 502
Ashburner, C. A., on oil recovery, 214; on the anticlinal theory, 219; on the geology of natural gas, 209
Atwood, W. W., on an old erosion surface in Idaho, 120
Auric oxide, 163
Austin, C. L., analysis by, 613
Auvergnose, analysis of, 187

Bain, H. F., on determination of pore space in rocks, 215
Baker, M. B., on the geology of Kingston, 201
Balcones fault, 478
Barrell, Joseph, on pegmatites in Montana, 189
Barriefield, Ont., quaquaversal structure, 202
Basalt, Korea, 8
Bastin, E. S., *War time mineral activities in Washington*, 524–537; on Cobalt ores, 385; on the pegmatites of Maine, 189
Bateman, Alan M., *A tungsten deposit near Fairbanks, Alaska*, 112–115; discussion by, 222
Batholith, Suan Mining Concession, 3
Bauxite, 502
Becker, G. F., on lateral pressure, 482; on the force of growing crystals, 461
Bedrock deposits of road materials, 560
Bendigo gold field, geology, 101
Bendigo quartz veins, Replacement in, and its relation to gold deposition (Stillwell), 100
Bendigo quartz veins, origin, 538
Bergeat, Alfred, on the Aranzazu zone, 259
Bibliography, oil migration, 347
Birch Creek schists, 113
Bird, R. M., and Calcott, W. S., on the association of vanadium with petroleum and asphalt, 57
Bismuth, 502
Blackwelder, E., on geology of Liaotung Peninsula, 9; on the age of the Idaho-Washington peneplain, 121
Blake, W. P., on the Deep Creek district, Utah, 249

Blanchard, A. A., on the cone specific gravity method, 586
Blatchley, R. S., on the occurrence of oil in Illinois, 211
Blebs, use of term, 97
Block diagrams of topography, use in war, 146
Bomb of chrome vanadium steel, 168
Bore holes, temperature measurements in, 278
Bowen, N. L., on the evolution of igneous rocks, 192, 196
Bownocker, J. A., on porosity of Berea grit, 322
Brecciation in quartz, 104
Broderick, T. M., *Some features of magnetic surveys of the magnetite deposits of the Duluth gabbro*, 35–49
Brooks, A. H., letter concerning J. D. Irving, 554
Buckley, E. R., on determination of pore space in rocks, 215
Building stones and clays (Richardson), review of, 70
Bulkley intrusives, 352
Burgess, J. A., discussion by, 546; on origin of haloid salts at Wonder, Nev., 224
Butte district, physiographic features, 131

Calcification of wood, 116
Calcite, 360
Calcite. The replacement of wood by (Greenland), 116
Cameron, F. K., on solubility of gypsum, 618
Campbell, M. R., on the occurrence of oil in rocks, 212; on theories of oil accumulation, 316
Camptonose, analysis of, 187
Cane Springs mining claim, 248, 257, 272
Cap rock of salt domes, origin, 616
Capillary concentration theory of oil accumulation, 319
Capillary interstices, 208
Capillary pressure of oil and water, 340
Capital Issues Committee of the Federal Reserve Board, 535
Carbonates, insoluble, action toward gold solutions, 164
Carnegie swamp or trench, 599
Carnotite, 50, 51; publications on, 50; solubility, 57; synthesis attempted, 58
Carnotite deposits, 52; at Mauch Chunk, Pa., 53
Cavities in oil sands, 207; in rocks, 320
Cementation, 342
Cementation test of rock, 566, 573, 577
Ceylon graphite, 507
Chalcocite, 359, 364, 369; origin in Engels ores, 91
Chalcopyrite, 359, 365
Chalk, 502
Chamberlin, R. T., and Miller, W. Z., on low-angle faulting, 606
Chañarcillo Mines, Chile, haloids of, 225
Chance, H. M., on estimating underground oil resources, 214
Chemical analyses—granite, average composition, 143; ilmenite, 422, 424, 434; magnetite, 437; nickel eruptive, 143; olivine gabbro, 187; pegmatitic gabbro, 187; racewinite, 613; sudburite, 143
Chemical causes of oil migration, 337, 342
Chemical experiments bearing on the origin of certain uranium-vanadium ores (Notestein), 50
Chester Hill, Mass., ilmenite from, 438
Chil Sing Dei granite, 7
Chloritic quartz, 106
Chromite, sources, 503
Clapp, C. H., review by, 400
Clapp, F. G., on the occurrence of oil, 211; on the occurrence of natural gas in Ontario, 203
Clark, K. A., and Reinecke, L., on sampling deposits of road materials, 561
Clark, W. O., on estimating underground water resources, 214
Clarke, F. W., on haloids in sea water, 224; on pseudo-serpentine from Stevens Co., Wash., 381; on the occurrence of vanadium, 52
Classification, causes of dips, 604; of oil migration, 337, 343
Clay, imported and domestic, use of, 504; literature on, 76, 240
Cleland, H. F., discussion by, 145
Clements, J. M., *Petroleum in Japan*, 512–523
Clendenin, W. W., on the origin of salt domes, 455
Clifton Hills, Utah, 249; mining activity, 247

Coal, literature on, 74, 238
Cobalt, Ont., ore deposits, 385
Coleman, A. P., average analysis by, 143
Collbran contact of the Suan Mining Concession, Korea, Geology and ore deposits of (Higgins), 1
Collbranite, 19
Colorimetric tests of for organic impurities in sand, 589
Columbia River basalt, 488
Comcar formation, 281
Committee on Mineral Imports and Exports, 530
Committee on Public Information, 536
Concentration of oil and gas, 344
Contact metamorphism, Suan Mining Concession, 16
Contact-metamorphic deposits, Gold Hill district, Utah, 271
Contact-replacement deposits, Gold Hill district, Utah, 273
Contacts, Tul Mi Chung, 4
Contour map of dip needle deflections, 38
Copper, as a war mineral, 505; literature on, 71, 233; Telkwa district, B. C., 355
Copper ore, Gold Hill district, Utah, 262, 270
Copper silver veins of the Telkwa district, British Columbia (Dolmage), 349
Cost of macadam road, 560
Coste, Eugene, on natural gas in Ontario, 209; on the origin of salt domes, 455
Craig, E. H. C., on the accumulation of oil, 211; on the action of oil on reservoir rocks, 216
Crawford, W. G., experiments on gold hydroxide, 172
Creep in strata, 609
Crevices and cavities in oil sands, 207
Crew, B. J., on the occurrence of oil, 210
Crocidolite, 502
Crook and Jones, analysis of picroilmenite by, 428
Crook, T., and Blake, G. S., on carnotites, 50
Crosby, W. O., and Fuller, M. L., on pegmatites in Massachusetts, 189
Crossbedding, 602
Cryolite, 505
Cumberlandite, 432
Curves of mineralization, Suan Concession, 21
Cushing, H. P., on superficial folds, 200
Cushman, A. S., on the effect of water on rock powders, 573
Cyanite, 256

Dale, T. N., on dikes of pegmatite in Vermont, 189
Daly, M. R., on static friction in rocks, 217; on the diastrophic theory, 220
Daly, R. A., average analysis by, 143; on gases in rock crystallization, 85; on pegmatites in British Columbia, 189; on underground volatile agents, 144
Darwin, G. H., on variations in vertical due to elasticity of earth's surface, 217
Dawson, G. M., on the Eocene age of the Idaho-Washington peneplain, 123
Deep Creek Valley, Nevada, 251
DeGolyer, E., *The significance of certain Mexican oil field temperatures*, 275–301; discussion by, 616; on fissures in oil rocks, 212; on the origin of salt domes, 454
Deposition of gold in nature, Further studies on (Lenher), 161
Deposition of strata on uneven floor, 209
Deussen, Alex., on salt domes, 449
Diagrams—andalusite crystal, 256; mineralization, Suan Concession, 20; occurrence of natural gas in Ontario, 203; quaquaversal structure at Barriefield, Ont., 202
Diapir fold, 467
Diemer, on the reduction of auric chloride, 174
Differentiation of pegmatites, 196
Dike, pegmatite, 189
Diopside, 18
Diorite, 6
Dip, original, causes of, 200
Dip needle deflections, contour map of, 38
Dips, Anomalous (Shaw), 598
Discussion—
Crevices and cavities in oil sands (Shaw), 207
Genetic classification of underground volatile agents (Emmons), 144
The geologist in war time—geol-

ogy on the Western Front (Cleland), 145
The geologist in war times: relation to military water supplies (Meinzer), 314
The geologist in war times: The United States Geological Survey's war work (Smith), 392
Halogen salts of silver at Wonder, Nevada (Burgess), 546; (Young), 224
The Idaho peneplain (Lindgren), 486; (Livingston), 488
Intrusive origin of the Gulf Coast salt domes (Jones), 621
Magmatic ore deposits of Sudbury, Ontario (Goodchild), 137
Movements of oil and gas through rocks (Washburne), 550
The occurrence of the halogen salts of silver (Lindgren), 225
Origin of the Bendigo quartz veins (Taber), 538
Origin of the cap rock of the Gulf Coast salt domes (DeGolyer), 616
Occurrence of the silver halides in the oxidized zone of ore deposits (Knopf), 622
The petrology of reservoir rocks and its influence on the accumulation of petroleum (Jones), 147; (Rogers), 316
Petrology of reservoir rocks (Lewis), 65
Replacement in the Bendigo quartz veins (Bateman), 222
Silver deposition and enrichment at Cobalt, Ont. (Reid), 385
Dobie, Walter, on negative magnetic lines, 41
Dolmage, V., *The copper silver veins of the Telkwa district, British Columbia*, 349–380
Dolomite, value for road building, 579
Dome structure in Ontario, 203
Domes, salt, origin of cap rock, 616
Dorsey, Hager, on faulting in oil fields, 211
Drilling in Japanese oil fields, 513
Duluth gabbro, Some features of magnetic surveys of the magnetite deposits of the (Broderick), 35
Duluth gabbro, The pegmatites of (Grout), 185
Dumble, E. T., on the occurrence of petroleum, 453
Dunn, E. J., on the Bendigo gold field, 103, 539
Durham, H. W., on street paving, 580
Dutch Mountain, Utah, geologic column, 263

Echigo oil district, Japan, 513, 515
Economic geology, principles of, 325
Economic geology, recent literature on, 71, 233
Egersund, Norway, ilmenite from, 434
Emmons, S. F., on Cobalt ore deposits, 387
Emmons, W. H., discussion by, 144; review of book by, 325; on enrichment of ore deposits, 390
Emmons, W. H., and Harrington, G. L., on ground waters, 56
Engels, California, Further remarks on the ores of (Graton and McLaughlin), 81
Enrichment at Cobalt, Ont., 385; at Superior mine, 92
Erosion surface in Idaho: is it Eocene? (Rich), 120
Excelsior claim, Telkwa district, B. C., 356, 358
Experiments on gold hydroxide, 172; on salt crystals, 474; on uranium and vanadium, 56; to show distortion of earth's magnetic field, 45; with Iceland spar and gold chloride, 175
Experiments bearing on the origin of certain uranium-vanadium ores, Some chemical (Notestein), 50
Export conservation list, 531

Fairbanks, Alaska, A tungsten deposit near (Bateman), 112
Fairbanks district, Alaska, physical features, 113
Faulting, Bendigo gold field, 106; Llano-Burnet region, Texas, 479
Federal Reserve Board, 535
Federal Trade Commission, 528
Fenneman, N. M., on salt domes, 449
Fenner, C. N., on magmatic differentiation, 195
Fissuring in oil fields, 323
Fleck, H., and Haldane, W. G., on origin of carnotite deposits, 55
Flint-grinding pebbles, 505
Flow structure in salt, 470
Flowage in strata, 608

Folding, superficial, 200
Free, E. E., on dry sands, 216
Frontenac County, Ont., molybdenite deposits, 312
Fuller, M. L., on estimating underground water resources, 215
Fuller's earth, 506
Furnace, heating, construction, 169
Further remarks on the ores of Engels, California (Graton and McLaughlin), 81
Further studies on the deposition of gold in nature (Lenher), 161

Gabbro, analyses of, 187
Gale, H. S., on origin of carnotite deposits, 55
Galena, 359
Galloway, J. D., on chalcopyrite, 376
Gardner, J. H., on local folding, 606
Gas, in Japanese oil districts, 518; movement through rocks, 550
Gas pressure, 343
Gems, literature on, 76, 240
Genesis of dips, 604; of Suan deposits, Korea, 12, 19; of Telkwa district ore deposits, 361, 368 (*see also* Origin)
Genetic classification of cause of dips, 604
Geologic maps—Clifton (Deep Creek) district, Utah, 261; Gold Hill district, Utah, 261; Gold Hill mine, 264; Suan Mining Concession, 3 (*see also* Maps)
Geology, Allen Co., Ky., 204; Bendigo gold field, 101; Clifton district, Utah, 251; Ely, Nevada, 400; Japanese oil fields, 515; northeastern Mexico, 280; Pontiac Co., Quebec, 302; salt dome region, 478; Telkwa district, B. C., 350
Geology and ore deposits of Ely, Nevada, review of, 400
Geology and ore deposits of the Collbran contact of the Suan Mining Concession, Korea (Higgins), 1
George, R. D., on pegmatites in Colorado, 193
Geothermal gradient, 297
Gidel, M. H., experiments on copper formation, 270
Gilbert, G. K., on superficial folds, 200
Glacial boulder deposits, for road making, 580
Gneiss, tests on, for road building use, 578
Goesmann, C. A., and Buck, C. E., on the origin of salt domes, 456
Gold, deposition of, in nature, Further studies on (Lenher), 161
Gold, literature on, 72, 234; occurrence in Bendigo field, 108; native, 258
Gold deposition, Replacement in the Bendigo quartz veins and its relation to (Stillwell), 100
Gold deposits of the Rand (Horwood), review of, 150
Gold hydroxide, 165; behavior when heated, 166, 167; experiments on, 172
Gold oxide, 163
Gold solutions, effect of insoluble carbonates on, 164
Gold Hill, Utah, geologic column, 263; mine, 262
Gold Hill and vicinity, Tooele County, western Utah, Notes on (Kemp and Billingsley), 247
Goodchild, W. H., discussion by, 137
Granodiorite, 260
Graphic structure in ores, 97
Graphite as a war mineral, 506
Graton, L. C., and McLaughlin, D. H., *Further remarks on the ores of Engels, California*, 81–99
Gravel, abrasion tests, 588; influence of texture in road construction, 590; specific gravity determination, 587
Gravel deposits for road making, 583
Gravitative causes of oil migration, 338
Greenland, C. W., *The replacement of wood by calcite*, 116–119
Grenville series, Quebec, 302, 304
Grout, Frank F., *The pegmatites of the Duluth gabbro*, 185–197
Gulf coast salt domes, Intrusive origin of the (Rogers), 447
Gypsum, solubility, 619

Hager, Dorsey, on estimating underground oil resources, 214
Hager, Lee, on the origin of salt domes, 449, 455
Hahn, F. F., on the form of salt deposits, 454, 465
Halides, silver, in ore deposits, 622
Halogen salts of silver at Wonder, Nev., 224, 225, 546
Hardin, on silver hydroxide, 166
Hardness test of stone, 566, 573
Harker, A., on igneous rocks, 310

Harrington, analysis by, 437
Harris, G. D., on the origin of salt domes, 449, 456, 459
Hayes, C. W., on the action of organic matter on rocks, 216
Hayes, C. W., and Kennedy, Wm., on salt domes, 449
Hazelton formation, 350
Heat, of saline domes, source, 299; loss in oil wells, 299
Heat-measuring device, 170
Heating furnace, construction, 169
Heaving of Niagara limestone at Chicago, 601
Hematite, 365
Hess, F. L., on origin of carnotite ores, 55; on formula for carnotite, 51
Hickey, P. J., analysis by, 613
Higgins, D. F., *Geology and ore deposits of the Collbran contact of the Suan Mining Concession, Korea*, 1–34
Highland Boy mine, mineral from, 611
Highway engineering, 557
Hilgard, E. W., on the origin of salt domes, 455
Hill, R. T., on the origin of salt domes, 456
Hillebrand, W. F., on the occurrence of vanadium, 55
Hoden Oil Company, Japan, 514, 521
Hol Kol granite, 6
Hol Kol limestone, 4
Hopkins, O. B., on salt domes, 449
Hotchkiss, W. O., on magnetic surveys, 36, 40
Howe, Ernest, review by, 227
Hubbard, L. L., on the geology of oil-bearing rocks, 210
Hubbard, P., and Jackson, F. H., Jr., on tests of road building rocks, 576
Hunt, T. S., on the anticlinal theory of oil accumulation, 209
Hunt, W. F., on sulphur associated with salt, 483
Hydrodynamic pressure, 339
Hydrothermal ore, 87
Hydroxide of gold, 165
Hydroxide of silver, 166
Hypabyssal, use of term, 605

Iceland spar and gold chloride, experiments on, 175
Idaho, relief map, 488
Idaho peneplain, 486, 488
Igneous activity, northeastern Mexico, 282
Igneous rocks, Suan Mining Concession, 5
Iki, T., on Echigo oil field, 516
Ilmenite, 420; from Arendal, Norway, 423; from Miask, Russia, 421; from Snarum, Norway, 425
Ilvaite, 19
Induced polarity of magnetic bodies, 46
Induced porosity of rocks, 317, 320
Induration of sediments, 603
Insizwa ore deposits, 140
Intergrowths of minerals, 421
Interstices in rocks, 208
Interstitial porosity, 317
Intratelluric causes of oil migration, 337, 342
Intrusion, mechanics of, Suan Concession, 14
Intrusive origin of the Gulf coast salt domes (Rogers), 447
Intrusive theory of salt dome origin, 467
Investigation of road materials, 559
Iodine, reactions in oxidizing zone, 623
Iron, literature on, 71, 233
Iron ore, 507
Iron Mountain, Wyo., ilmenite from, 429
Irving, John Duer (Lindgren), 413; letters concerning his work in France, 553
Iserine, 436

Jackson, F. H., Jr., on toughness test for rock, 571
Jenkins, O. P., *Notes on the possible origin of the magnesite near Valley, Washington*, 381–384
Johnson, D. W., on block diagrams, 146
Johnson, R. H., and Huntley, L. G., on oil reservoirs, 212
Johnston, John, and Adams, L. H., on properties of salt, 475; on temperature measurements in bore holes, 280
Joint Information Board of Minerals and Derivatives, 535
Jones, W. F., discussions by, 147, 621

Kemp, J. F., review by, 150
Kemp, J. F., and Billingsley, Paul, *Notes on Gold Hill and vicinity, Tooele County, western Utah*, 247–274

Kennedy, A. E., and Schurig, O., on the tractive power required on roads, 568
Kennedy, William, on the origin of salt domes, 455; on temperatures of oils in saline dome region, 296
King, F. H., on estimating underground water resources, 215
King vein, Telkwa district, B. C., 372
Knight, C. W., on Sudbury ore deposits, 139
Knopf, Adolph, discussion by, 622
Korea, Geology and ore deposits of the Collbran contact of the Suan Mining Concession (Higgins), 1
Korea granite, 9
Koto, B., on ilvaite, 19
Kragerö, Norway, ilmenite from, 424
Krüss, on hydroxide of gold, 165

Laboratory tests of road materials, 565
Lachmann, R., on salt deposits, 480
Lake Sanford, N. Y., ilmenite from, 431
Lattice structure, origin, 96
Lauer, A. W., discussion of paper by, 147, 316; on oil and gas reservoir rocks, 207
Laurentian system, Quebec, 302, 303
Lea, M. C., on silver hydroxide, 166
Lead, literature on, 72, 235
Leith, C. K., *"War minerals" as a science*, 497–511
Lemhi County, Idaho, map, 126
Lenher, Victor, *Further studies on the deposition of gold in nature*, 161–184
Lerch, Otto, on the origin of salt domes, 455
Lesley, J. P., on anticlinal lines in Pittsburgh region, 219; on interstitial space in rocks, 209; on the mode of occurrence of petroleum in rocks, 210
Lewis, J. O., discussion by, 65
Limestone, value for road building, 579
Lindgren, Waldemar, *John Duer Irving*, 413–418; discussions by, 225, 486; on banded structure in quartz veins, 104; on origin of carnotite deposits, 55; on origin of salt domes, 460; on the Bendigo gold veins, 539
Lindgren, W., and Ransome, F. L., on pegmatites in Colorado, 189
Literature on economic geology, Recent (*see* Recent literature, etc.)
Livingston, D. C., discussion by, 488
Lucas, A. F., on the origin of salt domes, 455
Lucy L. mine, 260
Lyman, B. S., on petroleum in Japan, 513

Macadam road, cost, 560
McAlpin arch, 601
McCoy, A. W., on capillary pressure, 609; on local folding, 606
McLaughlin, D. H., with Graton, L. C., *Further remarks on the ores of Engels, California*, 81–99
Magmas, consolidation, 141
Magmatic, use of term, 84
Magmatic differentiation, 310
Magmatic ore deposits of Sudbury, Ont., 137
Magnesite, as a war mineral, 507
Magnesite and gold chloride solution, 177
Magnesite near Valley, Washington, Notes on the possible origin of the (Jenkins), 381
Magnet Cove, Ark., magnetite from, 437
Magnetic bodies with no attraction, 48
Magnetic surveys, methods, 36
Magnetic surveys of the magnetite deposits of the Duluth gabbro, Some features of (Broderick), 35
Magnetite, 437
Magnetite deposits of the Duluth gabbro, Some features of magnetic surveys of the (Broderick), 35
Magnetite ore, Telkwa district, B. C., 375
Magpie quartz, 105
Manganese as a war mineral, 497, 507
Maps—Clifton district, Tooele Co., Utah, location, 250; Cane Spring and Alvarado mines, Clifton district, Utah, 254; Duluth gabbro, 186; Lemhi County, Idaho, 126; Korea, 2; Suan Mining Concession, 3 (*see also* Geologic maps)
Marine limestone beds, departure from horizontal, 198
Martin, G. C., on the occurrence of petroleum, 211
Mather, K. F., *Superficial dip of marine limestone strata; a factor in petroleum geology*, 198–206; cited, 598
Meinzer, O. E., discussion by, 314
Mendez shale, 281
Mercury as a war mineral, 508

Merrill, G. P., on determination of pore space in rocks, 215
Mesothorium, 509
Metamorphic rocks, Korea, 8
Metals, minor, literature on, 73, 235
Methods of magnetic surveys, 36
Mexican oil field temperatures, The significance of certain (DeGolyer), 275
Mica as a war mineral, 508
Microphotographs (*see* Photomicrographs)
Microstructure of certain titanic iron ores, On the (Warren), 419
Migration of oil, 335; classification of causes, 337; of oil and gas, 220
Milch, L., on plasticity of crystals, 474
Military water supplies, 314
Mills, R. V. A., and Wells, R. C., on the origin of salt domes, 456
Milne, John, on earth pulsations, 217
Mineral Administrator bill, 537
Mineral data, collection of, 525
Mineral information, distribution of, 535
Mineral resources, literature on, 79, 242
Mineralization, definition, 21; Gold Hill district, Utah, 266; Suan Mining Concession, 16, 20; Telkwa district, 352
Minor metals, literature on, 73, 235
Mixter, on hydroxide of gold, 165
Moesta, Fr. A., on the occurrence of haloid salts of silver, 225
Molecular causes of oil migration, 339
Molybdenite, 5, 257
Molybdenite deposits, types of, 305
Molybdenite ores, A pegmatitic origin for (Thomson), 302
Monazite as a war mineral, 509
Moore, R. B., and Kithil, K. L., on origin of carnotite deposits, 54
Morey, G. W., on apparatus for heat experimentation, 167
Morris, W. W., with Lenher, V., Behavior of gold hydroxide when heated above 200° under pressure, 167
Mortar tests of sand, 588, 593
Mount St. Patrick molybdenite mine, 309
Movements of oil and gas through rocks (Ziegler), 335
Mrazec, L., and Teisseyre, W., on Roumanian salt deposits, 466
Mügge, O., on intergrowths of minerals, 420
Munn, M. J., on static friction in rocks, 217

Nash, J. P., on tests of concrete road aggregates, 589
National Research Council, 536
Natural gas, literature on, 74, 238
Negative dip needle deflections, 40, 42
Negative lines of polarity, 41
Niagara limestone at Chicago, heaving of, 601
Nicholas, W., on gold quartz reefs of Australia, 100
Nickel Commission of Ontario, Report of, review of, 227
Nippen Oil Company, Ltd., 514, 521
Nishiyama oil field, 514
Nitrates, 509
Non-bituminous road materials (Reineke), 557
Non-metals, literature on, 76, 241
Norton, E. G., on the origin of salt domes, 455, 608
Notes on Gold Hill and vicinity, Tooele County, western Utah (Kemp and Billingsley), 247
Notes on the possible origin of the magnesite near Valley, Washington (Jenkins), 381
Notestein, F. B., *Some chemical experiments bearing on the origin of certain uranium-vanadium ores*, 50–64

Oil, of Japanese fields, character, 518; movement through pores, 147; through rocks, 550
Oil, refineries in Japan, 519
Oil accumulation, cause, 336; bearing of origin of salt domes on, 621; in salt domes, 621
Oil and gas, the movements of, through rocks (Ziegler), 335
Oil and gas migration, 220
Oil migration, factors conditioning, 346; list of papers on, 347
Oil field temperatures, Mexican, 274
Oil sands, crevices and cavities in, 207
Oil shales, literature on, 76, 240
Oil temperatures, 283
Olcott, E. E., specimens collected by, 247
On the microstructure of certain titanic iron ores (Warren), 419
Onslow molybdenite deposits, 305

Openings in sedimentary rocks, 221
Ore deposits, Suan mine, 10
Ores of Engels, California, Further remarks on the (Graton and McLaughlin), 81
Organic impurities in sand, testing of, 589
Origin of Bendigo quartz veins, 538; cap rock of Gulf coast salt domes, 616; 483; carnotite deposits, 54; Cobalt, Ont., ore deposits, 390; copper deposits, Gold Hill district, Utah, 270; Gulf coast salt domes, 447, 454; halogen salts of silver at Wonder, Nevada, 546; lattice structure, 96; magnesite near Valley, Wash., 381; magnetites of Duluth gabbro, 35; molybdenite ores, 302; oil, 336; quartz reefs, 103; Rand gold deposits, 150; silver ore at Wonder, Nev., 224, 225; Sudbury magmatic ore deposits, 138; Sudbury nickel deposits, 230; tungsten deposit, Alaska, 115 (*see also* Genesis)
Origin of certain uranium-vanadium ores, Some chemical experiments bearing on the (Notestein), 50
Orthotectic, definition of term, 85
Oscillating water table, 144
Owen, R., on the origin of salt domes, 455

Pabst, V., on the Bendigo gold field, 100
Pack, R. W., on character of oil-bearing rocks, 213
Page, L. W., on road materials, 564, 569
Page, Sidney, on faulting in Texas Paleozoic, 479
Paragenesis, Suan ores, 13, 22; Telkwa district, B. C., 354, 360, 367
Pascoe, E. H., on temperatures of oils in Burma, 295
Payette lake beds, 487
"Peace minerals," 499
Pebbles, flint-grinding, 505
Pegmatite separation, mechanics of, 193
Pegmatites, differentiation, 196
Pegmatites of the Duluth gabbro (Grout), 185
Pegmatitic origin for molybdenite ores (Thomson), 302
Pelletier, J., on the preparation of auric hydroxide, 163
Peneplain, Eocene, in Idaho, Montana, and Washington, 121
Permian deposits of Texas, 476
Petroleum, literature on, 74, 238; production in Japan, 522; reservoir rocks, 65 (*see also* Oil)
Petroleum in Japan (Clements), 512
Petrography, molybdenite deposits, 306
Petrology of reservoir rocks, 65
Phlogopite, 17
Phosphate, literature on, 76, 240
Photomicrographs—copper ores, 358; gold on Iceland spar, 176; hematite in ilmenite, 446; magnetite-ilmenite, 446; slate altering to andalusite hornfels, 255; Squaw Lake titaniferous rock, 310; titanic-iron ore, 443, 445; washingtonite, 442
Physiographic history of the northern Rocky Mountain region, 121
Picro-ilmenite, analysis of, 428
Pig Tin Committee, 531
Placers, Korea, 4
Plasticity of salt, 473
Platinum, 510
Plevna molybdenite deposit, 312
Pneumatolytic, use of term, 85
Pneumatolytic action, 82
Pneumotectic, definition of term, 85
Polarity of magnetic bodies, 46
Polymerization of oil, 342
Pond, W. F., study of titanium minerals, 419
Popular oil geology, review of, 493
Pore space of rocks, estimating, 215
Porosity of rocks, 317; of sandstones, induced, 66; relation to oil movement, 147
Potrero de Llano well, 299
Pre-Cambrian rocks, Korea, 9
Principles of economic geology, review of, 325
Prindle, L. M., and Katz, F. J., on the Birch Creek schists, 113
Priority in rail transportation, 534
Production of war minerals, 525
Proterozoic, Korea, 9
Purdue, A. H., on the origin of sink holes, 602
Pyrite, 510

Quaquaversal structure at Barrie-field, Ont., 202
Quartz reefs, Bendigo, 102
Quicksilver as a war mineral, 508
Quyon molybdenite mine, 306

Racewinite, a peculiar mineral from ore deposits in Utah (Winchell), 611

Rainbow vein, Telkwa district, B. C., 377
Rammelsberg, analyses by, 427; on composition of ilmenite, 421
Rand gold deposits, 150
Randolph, Isham, on heaving of Niagara limestone at Chicago, 601
Ransome, F. L., on carnotite deposits, 53
Recent literature on economic geology, 71, 233
Recrystallization, 21
Red marble quarry, near Valley, Wash., 382
Red River fault, 478
Redwood, Boverton, on the Baku oil field, 211
Reefs, quartz, Bendigo, 102
Reeve, C. A., and Lewis, R. W., on toughness of bituminous aggregates, 579
Reeves, Frank, on dry sands, 216
Regional geology, literature on, 78, 242
Reid, J. A., discussion by, 385
Reinecke, L., *Non-bituminous road materials*, 557–597
Relief map—Idaho, 488
Replacement, selective, 12; in Bendigo gold field, 103
Replacement in the Bendigo quartz veins and its relation to gold deposition (Stillwell), 100; cited, 222
Replacement of wood by calcite (Greenland), 116
Replacement deposits, 266, 268; Telkwa district, B. C., 356
Report of the Royal Ontario Nickel Commission, review of, 227
Research Committee of the National Research Council, 536
Reserves of minerals, 526
Reservoir rocks, petrology of, 65
Resorption in Suan Mining Concession, 16
Reusch, E., on the plasticity of salt, 473
Reviews—
Building stones and clays (Richardson), Pirsson, 70
The geology and ore deposits of Ely, Nevada (Spencer), Clapp, 400
The gold deposits of the Rand (Horwood), Kemp, 150
Popular oil geology (Ziegler), Tomlinson, 493
The principles of economic geology (Emmons), Singewald, 325
Report of the Royal Ontario Nickel Commission, Howe, 227
Rhyolite flow, Deep Creek Valley, 252
Rich, J. L., *An old erosion surface in Idaho: is it Eocene?*, 120–136
Richardson, C. H., review of book by, 70
Richardson, G. B., on structure maps, 599
Rickard, T. A., on the Bendigo quartz veins, 100, 223, 545
Ries, H., on faulting in oil-bearing rocks, 211
Rinne, F., on the plasticity of salt, 473
Road materials, Non-bituminous (Reineke), 557
Roads, effect of traffic on, 562; service tests of, 564
Robertson, W. F., on hematite, 378
Rock pressure, 338
Rocky Mountain region, northern, physiographic history, 121
Rogers, G. S., *Intrusive origin of the Gulf coast salt domes*, 447–485; discussion by, 316
Roumania, salt deposits, 466
Rutile-ilmenite pebbles, 437

Saddle reefs, 538; Bendigo, 102
St. Urbain, Quebec, ilmenite from, 433
Salifère, 466
Salmon River, 129
Salt, character in Louisiana domes, 469; occurrence in Permian rocks, 476; plasticity, 473; use in drawing wire, 475
Salt deposits, Roumania, 466
Salt domes, distribution and relations, 450; Europe, 464; oil accumulation in, 621; origin of cap rock, 616; source of heat, 299
Salt plugs, forms of, 448
Salt-water temperatures, 284
Sampling road materials, 566
Sand, mortar tests, 588; testing organic impurities, 589
Sandstone, value for road making, 579
San Felipe beds, 281
San Fernando formation, 281
San Pedro formation, 281
Scheelite, 114
Schottländer, on hydroxide of gold, 165
Schünemans, F., on salt domes, 465

Scientific notes and news, 80, 158, 244, 333, 410, 494, 552, 625
Searles, Fred, Jr., letter concerning J. D. Irving, 555
Secondary concentration, Suan Concession, Korea, 10
Sections—calcified gymnosperm, 118; Tul Mi Chung mine, 4
Seoul Mining Company, 2
Service Bureau of the Committee on Public Information, 536
Service tests of roads, 561, 564
Settling of strata, uneven, 607
Seubert, K., and Rohrer, R., on halide salts, 622
Shaler, N. S., on dry sands, 216
Shaw, E. W., *Anomalous dips*, 598–610; discussion by, 207; on quantity of oil in Carlyle fields, 214; on static friction in rocks, 217; on the gravity of Gulf coast sediments, 481
Shaw, E. W., and Mather, K. F., on the geology of Allen Co., Ky., 204
Shedd, Solon, on magnesite of Washington, 381
Shipping Board, 530
Shooting of wells, effects, 218
Shore deposits, character and duration, 604
Shortening of strata, Suan Concession, Korea, 14
Sieves, Tyler system, 586
Significance of certain Mexican oil field temperatures (DeGolyer), 275
Silication, 27
Silicification, Suan mine, 24
Sillimanite, 256
Silver, haloid salts at Wonder, Nev., 224, 225; literature on, 72, 234; Telkwa district, B. C., 355
Silver deposition at Cobalt, Ont., 385
Silver haloids, origin, 546
Silver hydroxide, 166
Singewald, J. T., Jr., review by, 325; on ilmenite, 430; on intergrowths of minerals, 421
Sink holes, relation to dip, 602
Siroo Pong San gneiss, 4, 9
Slate altering to andalusite hornfels, 255
Slumping of strata, 199
Smith, P. S., discussion by, 392
Smyth, H. L., on disturbances of the earth's field by iron ore bodies, 40
Snarum, Norway, ilmenite from, 425
Sodium nitrate, 510
Solubility of gypsum in salt solutions, 618
Some chemical experiments bearing on the origin of certain uranium-vanadium ores (Notestein), 50
Some features of magnetic surveys of the magnetite deposits of the Duluth gabbro (Broderick), 35
Somers, R. E., *Recent literature on economic geology*, 71, 233
Sosman, R. B., on polarized magnetite, 43
Specific gravity determinations of gravel, 587
Specifications for road materials, 592
Spencer, A. C., review of book by, 400
Spiegeleisen, 508
Spurr, J. E., *War minerals*, 500–511
Squaw Lake molybdenite deposit, 308
Statistics of war minerals, collection of, 526
Stepovitch tungsten property, Alaska, 112
Stevens, Blamey, on intrusive pressure of mineralizing solutions, 103
Stillwell, F. L., *Replacement in the Bendigo quartz veins and its relation to gold depositions*, 100–111; cited, 538
Stone, R. W., on magnesite deposits of Washington, 381
Stresses, causes of, 608
Structure, salt domes, 452; Suan Mining Concession, 5
Structure maps, 599
Suan granite, 5, 7
Suan Mining Concession, Korea, Geology and ore deposits of the Collbran contact of (Higgins), 1
Suan slates, 4
Subcapillary interstices, 208
Subsidence, uneven, 607
Sudburite, 142; analysis of, 143
Sudbury magmatic ore deposits, 137; nickel deposits, 227
Supercapillary interstices, 208; space in oil-bearing rocks, 213
Superficial dip of marine limestone strata; a factor in petroleum geology (Mather), 198
Superficial folding, 200; slumping of strata, 199
Superior mine, ore deposits, 92
Surface tension phenomena, 340
Surveys, magnetic, methods, 36

Taber, Stephen, discussion by, 538
Tables—experiments on carnotite, 57; experiments on the effect of

heat and pressure on gold hydroxide, 174; on gold chloride, 181; oil temperatures, 283; paragenesis of minerals, Suan mines, 22; solubility of calcium sulphate in aqueous solutions of sodium chloride, 619; specifications for sand and gravel for roads, 592; temperatures in bore holes, 283
Tamasopo limestone, 280
Telkwa district, British Columbia, The copper silver veins of (Dolmage), 349
Telluride of gold, 260
Temepache formation, 281
Temperature of Mexican oil, relation to production, 274
Temperature measurements in bore holes, methods of, 277
Tertiary of Japan, 516
Test of cementing value of stone, 566
Testing road surfaces, 564
Texture of gravels, determination of, 585
Thermometers, used in oil temperature measurements, 278
Thomassy, R., on the origin of salt domes, 455
Thompson, A. B., on fissuring in oil-bearing rocks, 212; on oil and water temperatures, 295
Thomson, E., *A pegmatitic origin for molybdenite ores,* 302–313
Thorium, 509
Thornton, W. M., chemical analysis by, 422
Thorpe, Edward, on iodine in sea water, 225
Tin as a war mineral, 510
Titanic iron ores, On the microstructure of certain (Warren), 419
Tolman and Rogers, on magmatic sulphide ores, 84
Tomlinson, C. W., review by, 493
Tonalite-porphyry, 7
Tong Am limestone, 5; quartz-diorite dike, 5
Topography, Telkwa district, B. C., 350
Toughness test of stone, 565, 569
Tracy, Evarts, letter concerning J. D. Irving, 556
Traffic, effect on roads, 562
Tul Mi Chung limestone, 5
Tungsten as a war mineral, 511
Tungsten deposit near Fairbanks, Alaska (Bateman), 112
Turrentine, J. W., on potassium salts in salines, 456
Tyler system of sieves, 586
Tyuyamunite, 51

Udden, J. A., on salt in Permian rocks, 476
Umpleby, J. B., on an old erosion surface in Idaho, 120
Unconformities, 199
Underground solution, irregular, 609
Underground temperatures, 285
United States Geological Survey's war work, 392
Unjinsan batholith, 7
Uplift, uneven, 607
Uranium, experiments on, 56
Uranium-vanadium ores, Some chemical experiments bearing on the origin of certain (Notestein), 50

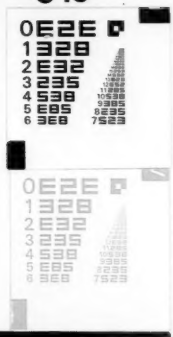
Valley Camp swamp, 600
Vanadium, as a war mineral, 511; experiments on, 56
Van der Gracht, W. A. J. M. von Waterschoot, on salt domes, 454
Vaughan, T. W., on crossbedding, 602
Vaughan, T. W., and Shaw, E. W., on induration of sediments, 604
Veatch, A. C., on salt domes, 449, 455
Vein formation, 540
Vesuvianite, 257
Viscosity of oil, relation to heat, 300
Voids test, 586, 592

War Finance Corporation, 534
War Industries Board, 528, 529
War Minerals (Spurr), 500
"War minerals" as a science (Leith), 497
War minerals, requirements, 527
War Minerals Committee, 536
War time mineral activities in Washington (Bastin), 524
War Trade Board, 526
Warner, C. A., calcified wood collected by, 116
Warren, Charles H., *On the microstructure of certain titanic iron ores,* 419–446
Warren district, Arizona, mineralization, 266
Warwick, N. Y., ilmenite from, 427
Washburne, C. W., discussion by, 550; on capillary concentration of oil and gas, 149, 349; on the origin of salt domes, 456, 460

Washingtonite, 426
Water supply for military purposes, 314
Water table, oscillating, 144
Watson, T. L., and Taber, S., on ilmenite from Virginia, 422
Weed, W. H., and Barrel, J., on pegmatites in Montana, 189
Weigall granite, 5
Wessely, Joseph, on dry sands, 216
Wherry, E. T., on calcification of wood, 119; on the occurrence of carnotite, 53
White, I. C., on oil recovery, 214
Williams, G. H., on origin of pegmatites, 185
Wilson, M. E., on contact pyroxenite, 304
Winchell, A. N., *Racewinite, a peculiar mineral from ore deposits in Utah*, 611–615
Winchester, D. E., on cross bedding, 603
Wood, The replacement of, by calcite (Greenland), 116
Wrather, W. E., on oil temperatures in Gulf Coastal region, 296

Young, J. W., discussion by, 224
Yung Ning sandstone, 9

Ziegler, Victor, *The movements of oil and gas through rocks*, 335–348; review of book by, 493; on the movements of oil and gas through rocks, 550
Zinc, literature on, 72, 235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